ADDIS ABEBA SCIENCE AND TECHNOLOGY UNIVERSITY

(AASTU)

DIVERSION WEIR DESIGN PROJECT

FOR

(ASHER RIVER)

SUBMITTED TO: - COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING

BY: - FELEKE GURMECHA

MASTERS OF ENGINEERING PROJECT IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE DEGREE OF MASTERS OF ENGINEERING IN HYDRAULICS ENGINEERING

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May 2017

Approval Page

This **Meng.** Project entitled with **“Diversion Weir Design Project** “has been approved by the following examiners in partial fulfillment of the requirement for the degree of Master of Engineering **Meng.** In **Hydraulics Engineering**.

Members of the Examining Board:

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   Examiner Signature Date

2. ________________________ ____________ _____________
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   Dean, CACE Signature Date

ACKNOWLEDGMENT

I am thankful to my family especially my wife for their great support at all the time and for giving me a word of confidence every once in a while, without their support the project work would be impossible.

I would like to thank all Amhara Supervision and Design Enterprise Office, Staff in AASTU College of Architecture And Civil Engineering Department, National Metrology Agency Dangila station for provide me with related materials.

Finally I can’t forget my friends and classmates for gave me encouragement, valuable consultations throughout the work of this Project.

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ABSTRACT

The purpose of this design diversion head work structure is to present the final result. Hence, it discusses the analysis of hydrology, hydraulic and structural design of the headwork and also presents bill of quantity and cost estimation are stated. This Study involves collecting data from different sources and using flow magnitude estimation method identifying maximum irrigable area which is 140 hectares, analyzing data, and changing the rainfall data to runoff using complex hydrograph method. Finally, this project ensures the peak discharge which has been designed 230.7m^3/sec on the safest side. Ogee types of weir is selected in order to dissipate the higher energy due to higher discharge. In additional part of the project based on the peak discharge design components of each hydraulic structure, under sluice, head regulator, stilling basin and divide wall. The structural of weir safe due to different load conditions. Besides, Type I stilling basin is selected.

1. INTRODUCTION

1.1. Background

Diversion headwork provides an obstruction across a river, so that the level of the water is raised and water is diverted to the channel at required level. The flow of water in the canal is controlled by the canal head regulator. This increased water level helps the flow of water by gravity and the increasing the commanded area and reducing the water fluctuation in the river (Garge, 2005).

Head works are barriers across a river at the head of an off taking main canal. Head works can be either diversion head works or storage headwork (Asawa, 2008). Diversion head works, are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt free water with a certain minimum head in to the canal.

Storage head work is a barrier constructed across the river valley to form the storage reservoir. The water is supplied to the canal from the reservoir through the canal head regulator. This serve as multipurpose functions like hydroelectric power generation, fishery, flood control, etc (Garge 2005).

A study also indicated that one of the best alternatives to consider for reliable and sustainable food security development is expanding irrigation development on various scales, through river diversion, constructing micro dams, water harvesting structures, etc. (Lambisso, 2005)

Different irrigation techniques such as diversion structures, storage, pumped etc. can be used. Diversion headwork structures are engineering facilities built across rivers or canals to store water and/or divert it from its original course. Among these, low-head diversion structures are extensively used in irrigation projects to divert water to a canal from either a canal or a natural river by raising the water level upstream

1.2. Statement of the Problem

The presence of unpredictable and variable nature of rainfall, farmland scarcity, poor soil fertility, occurrences of plant and livestock diseases and crop pests, absence or low use of modern inputs make the practice low productive and traditional type. Due to these increase the water efficiency by constructing permanent structure like Dam and Diversion Headwork Structures etc.

1.3. Objective of the Project

1.3.1. General objective

The primary objective of this project was to design diversion weir for Asher River.

1.3.2. Specific Objectives

The specific objectives were:-

- To identify the most suitable site for the diversion weir
- Determine weir type and weir cross section
- To prepare detail design calculations and design drawings for the weir and the bill of quantities.

1.4. Significance of the project

Region is gifted with different natural resources, agro-ecologies, bio diversities and huge manpower. The economy of the region largely depends on survival agriculture, which is traditional and rain fall dependent. The region has great potential for surface and subsurface water resource. The proposed River is one of the surface resources at that area. Design diversion structure on this river to expand the previous (15ha) traditional irrigation system to increase land by replacing the traditional diversion structure to modern irrigation by constructing permanent structure.

1.5. Organization of the project

The project is organized in to five chapters. Chapter one deals with introduction that covers the general background, the problem statement, objectives of the project, significance of the project and organization of the project.

Chapter two deals with the literature review used. Chapter three covers material and methodology of the project. Chapter four discuss about each hydraulic structure. Chapter five deals with Conclusion and Recommendations

2. LITERATURE REVIEW

2.1. Introduction

Head works are barriers across a river at the head of an off taking main canal. Head works can be either diversion head works or storage headwork (Asawa, 2008). Diversion head works, are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt free water with a certain minimum head in to the canal.

Storage head work is a barrier constructed across the river valley to form the storage reservoir. The water is supplied to the canal from the reservoir through the canal head regulator. This serve as multipurpose functions like hydroelectric power generation, fishery, flood control, etc. (Garge 2005).

The components of a diversion head work are weir or barrage, under sluices, divide wall, fish ladder, canal head regulator, silt excluders/silt prevention devices, river training works(Garge, 2005).

Under sluices /scouring sluices are openings provided at the base of the weir or barrage. These openings are provided with adjustable gates. Normally, the gates are kept closed. The suspended silt goes on depositing in front of the canal head regulator. When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounting on about. The muddy water flows towards the downstream through the scouring sluices so the gates closed. But, at the period of flood, the gates are kept opened (Bibhabasu, 2012).

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator. It consists of a number of piers which divide the total width of the canal in to a number of spans which are known as bays. The pier consists of tiers on which the adjustable gates are placed. The gates are operated from the top by suitable mechanical device. A platform is provided on the top of the pier for the facility of operating the gates. Again some piers are constructed on the downstream side of the canal head to support the roadway (Bibhabasu 2012).

Functions of canal head regulator are; it regulates the supply of water entering the canal, it controls entry of silt in the canal, it prevents the river flood from entering the canal (Mohanty, 2012).

The divide wall is a long wall constructed at right angles in the weir or barrage; it may be constructed with stone masonry or cement concrete. On the upstream side, the wall is extended just to cover the canal head regulator and on the downstream side it is extended up to the launching apron (Mohanty, 2012).

2.2. Uses of Weirs

According to the good practice manual by (Charles Rickard, Rodney Day and Jeremy Purseglove 2003), weirs have been constructed and used in England for the following four fundamental reasons:

A. Water Level Management

Most of the weirs in England and Wales have been constructed with the primary aim of water level management. The impoundment of water is clearly a central function of weirs as by their very nature they raise water levels relative to downstream conditions.

This is often achieved by the construction of a weir with a long crest, such that water level variation is small in response to changing flow conditions the alternative is to have a gated weir that will allow regulation of water level. Side weirs are frequently used for water level management in navigable waterways

Weirs are also used to divert water into off-stream reservoirs or diversion channels, for flood defense purposes or as part of a water supply scheme.

B. Flow Measurement

Weirs also form the backbone of the national hydrometric system, which provides accurate discharge information to facilitate development planning, flood forecasting, planning and development of flood alleviation schemes, and water resources regulation.

Although any weir can be used to provide information on flow rates, weirs not specifically designed with this in mind are likely to provide only approximate data. In the last fifty years or so, a large number of weirs have been constructed with the sole purpose of monitoring flow conditions in rivers, mostly until recently aimed at low to moderate flow conditions, and not high flood flows.

Flow gauging weirs permit engineers and hydrometrics to calculate the discharge in a river reach, monitor it over time and, if real time monitoring is available, to issue flood warnings and to adjust flood control structures in response to changing conditions\

C. Channel Stabilization

In reaches of river where the channel gradient is steep, and where erosion is an issue, the increased water depths caused by impounding will slacken water surface slopes, reduce and regulate velocities and help to control erosion.

Such weirs are much more common in southern Europe than they are in England and Wales. In this context, weirs are also provided in a reach of channel that has been shortened, so that the gradient in the stream can be maintained at a stable value. Weirs can also be used to create a silt trap, thereby preventing or reducing siltation downstream.

For such use it must be remembered that the effectiveness of the weir will depend on regular removal of the trapped silt, and this will require safe and easy access to the weir for suitable plant and equipment.

2.3. Types of Weirs

A weir with a sharp upstream corner or edge such that the water springs clear of the crest is known as a sharp crested weir.

All the other weirs are classed as weirs not sharp crested. Sharp crested weirs are classified according to the shape of the weir opening such as rectangular weirs, triangular weirs or v-notch weirs, trapezoidal weirs and parabolic weirs.

Weirs not sharp crested are classified according to the shape of their cross-section, such as broad-crested weirs, triangular weirs and trapezoidal weirs.

Sharp crested weirs are useful only as a means of measuring flowing water.

Weirs not sharp-crested are commonly incorporated into hydraulic structures as control or regulation devices, with measurement of flow as their secondary function.

2.3.1. Labyrinth Weir

A labyrinth weir uses a trapezoidal-shaped weir wall geometry plan view to increase the weir length. They are versatile structures and can be modified to fit many applications.

2.3.2. Broad-crested Weir

A broad-crested weir is a flat-crested structure, with a long crest compared to the flow thickness. When the crest is "broad", the streamlines become parallel to the crest invert and the pressure distribution above the crest is hydrostatic. The hydraulic characteristics of broad-crested weirs were studied during the 19th and 20th centuries. Practical experience showed that the weir overflow is affected by the upstream flow conditions and the weir.

2.3.3. Combination Weir

The sharp crested weirs can be considered into three groups according to the geometry of weir:

a) The rectangular weir,

b) The V or triangular notch and

c) Special notches, such as trapezoidal, circular or parabolic weirs.

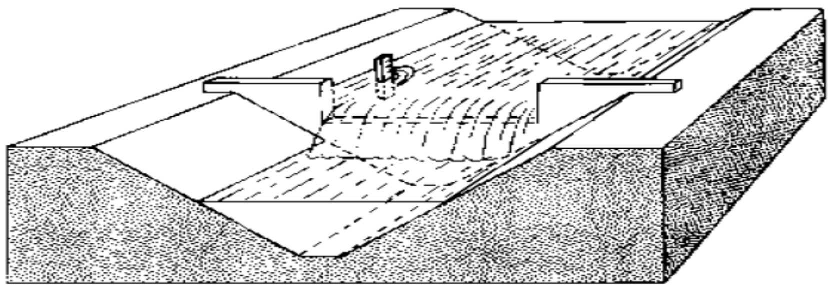


Figure 1: Rectangular Weir

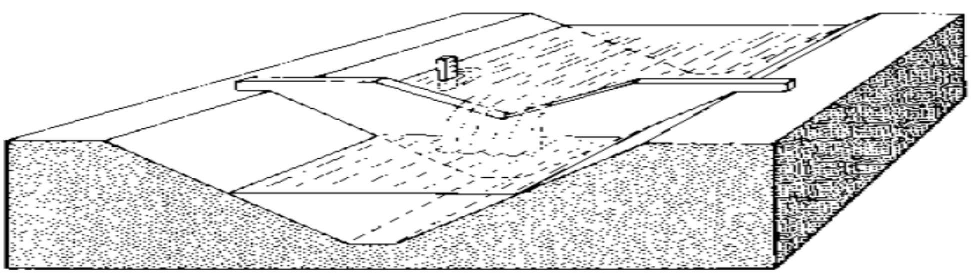


Figure 2: Triangular Weir or V-notch

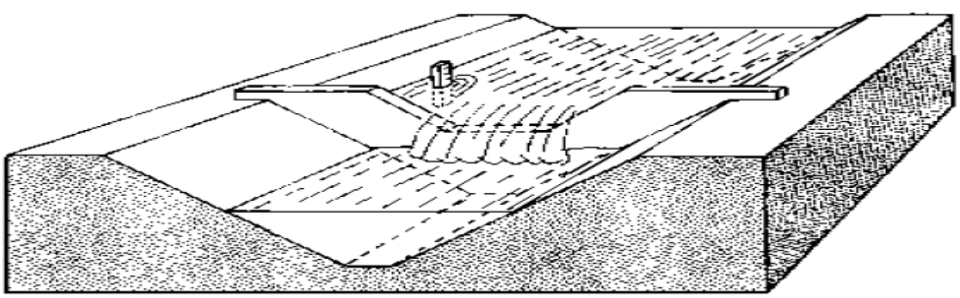


Figure 3: Trapezoidal Weir

2.3.4. V-notch Weir

The V-notch weir is a triangular channel section, used to measure small discharge values. The upper edge of the section is always above the water level, and so the channel is always triangular simplifying calculation of the cross-sectional area. V-notch weirs are preferred for low discharges as the head above the weir crest is more sensitive to changes in flow compared to rectangular weirs.

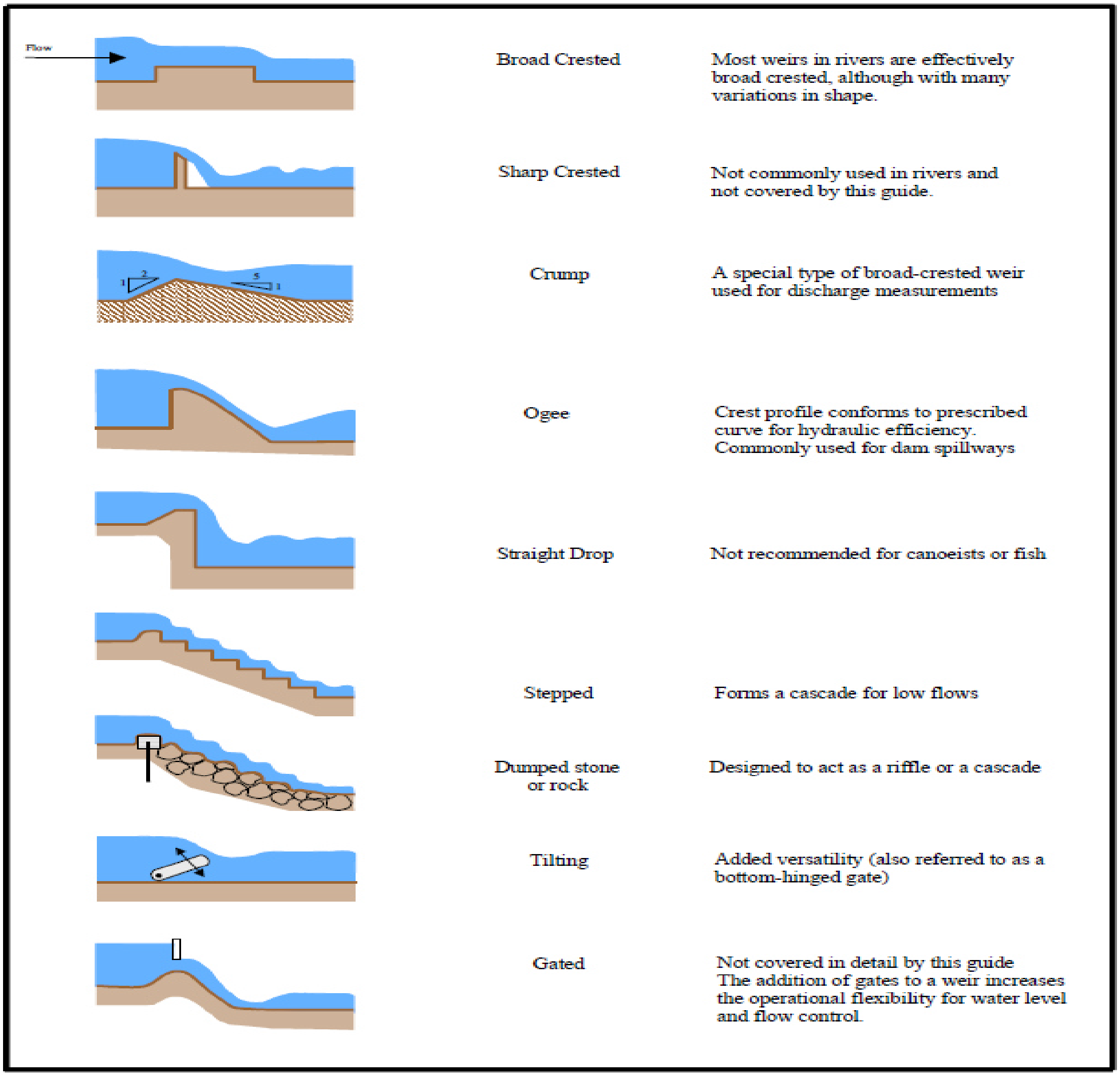


Figure 4: Weir type cross-sections (good practice manual by Charles Rickard, Rodney Day and
Jeremy Purseglove- 2003)

3. MATERIALS AND METHODS

3.1. Location and Description of the Study Area

The study area Asher Diversion Irrigation Project is located in Amhara Region, west Gojam zone. It is found 22 km from Merawi; 30 km from BahirDar. The project area is 6km from the woreda town Durbete. It is located at latitude 8 58′and longitude39 54′. The site has been visited for this project from 5-20 March 2016. It has not defined route but it is possible to construct new route. The area is characterized under Weynadega agro ecological zone. This project uses Asher Perennial River which is going to divert for irrigation purpose. Other water sources which are found around the project site are Andassa River, Abo spring, hand-dug wells and Anhisla River

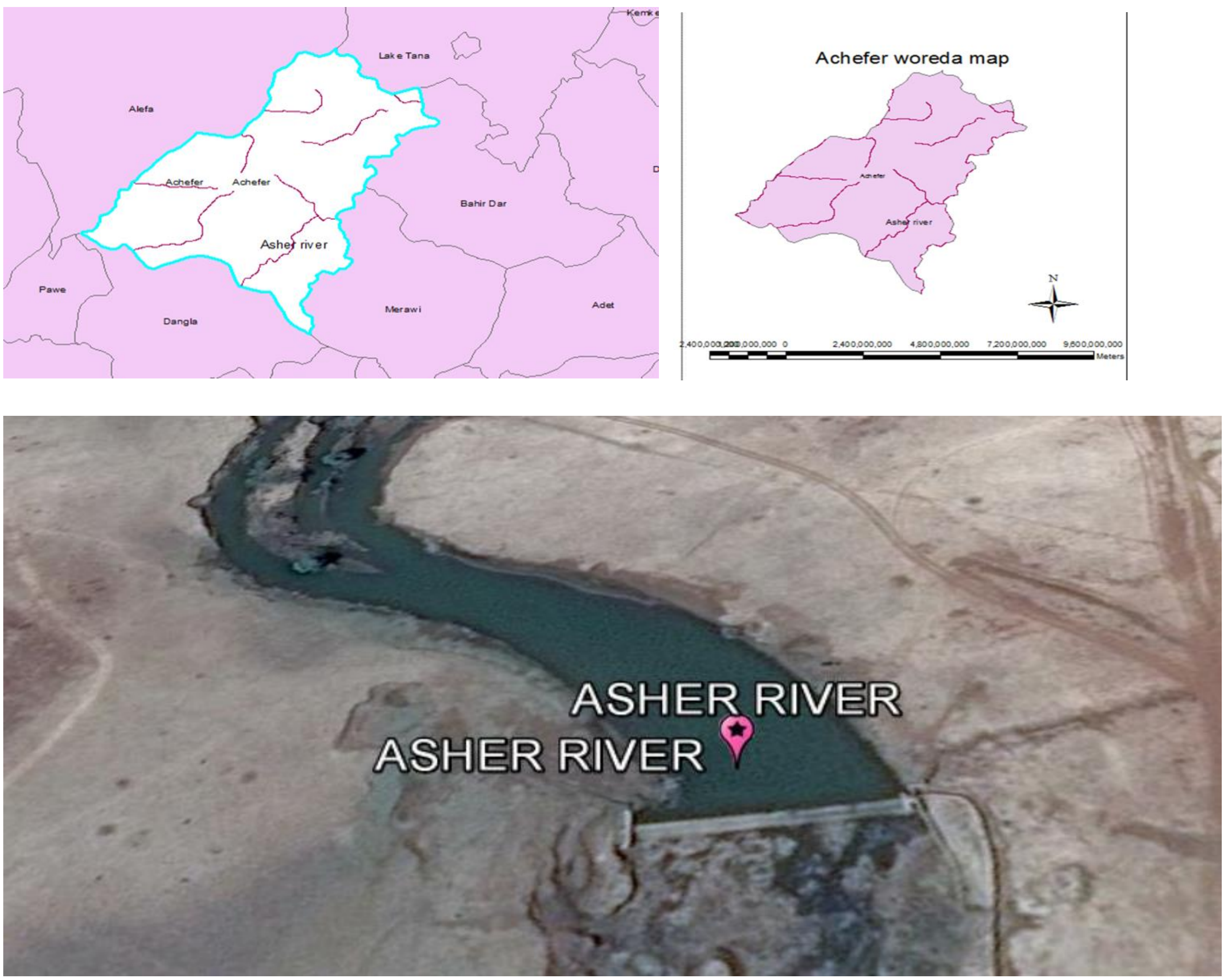


Figure 5: Location Map of the Asher River (source Google earth map and Arc GIS

3.2. Methodology

3.2.1. Data collection and Sample Size

The beneficiary or user community that is obtaining the services, field measurement and observation are the primary source of data for the design. In order to achieve the objectives of the design secondary data are also used. These data are obtained from Amhara water work construction enterprise and Amhara Design and supervision work that are found at grass root level. In addition to these literatures, different project documents or proposals, project evaluation and completion reports are also refereed.

A) Primary data collection

Field observation at Asher diversion weir design project was to identify where different parameters of the design head work must be taken. Availability of construction materials, irrigable area, geological characteristic, upstream and downstream of the head work, nature of the foundation, right and left abutment carefully selected points of the project was taken in collaboration with the Amhara water works construction enterprise workers.

B) Secondary data collection

Secondary data used for this project were collected from responsible bodies and officials. These data include climatic data which has Dangela Metrological station from 1997 to 2007 is taken 11 years of daily heaviest rainfall data is available for the design station, base flow or river discharge magnitude and water demand on the command area of the project.

Table 1: Sample Analysis for (source Dangila Meteorology Station)

S.No	Year	Heaviest rainfall (mm/day)	Descending Order	Rank	Log (Y)	(Yo-Ym)^2	(Yo-Ym)^3
1	1997	60	103.5	1	2.015	0.06	0.015
2	1998	49	84.4	2	1.926	0.024	0.004
3	1999	84.4	74.3	3	1.871	0.01	0.001
4	2000	66.9	66.9	4	1.825	0.003	0
5	2001	50	60	5	1.778	0	0
6	2002	48.3	60	6	1.778	0	0
7	2003	48.1	50	7	1.699	0.005	0
8	2004	74.3	49	8	1.69	0.006	-0.001
9	2005	103.5	48.3	9	1.684	0.007	-0.001
10	2006	60	48.1	10	1.682	0.008	-0.001
11	2007	37.3	37.3	11	1.572	0.039	-0.008
		Sum	**681.8**		19.52	0.16	0.01
		Mean	61.98		1.77	0.01	0.001
		STNDV	19.26		0.13	0.02	
		SKEW			0.4		

3.2.2. Data analysis

In the data analysis method first check the data consistency test of the daily heaviest rainfall data. These data should be checked for its consistency by outlier test.

$$\delta_n / \chi * 100 \quad \ldots\ldots\ldots\ldots \quad (3.1)$$

$$\sum (Y - \bar{Y})^2 = 0.16 \sum (Y - \bar{Y})^3 \quad \ldots\ldots\ldots\ldots \quad (3.2)$$

After checking the consistency of the data for both higher and lower outlier, the 11 years data obtained from the metrological station is taken as representative for the analysis.

$$C_s = \frac{N \sum (Y_i - \bar{Y})^3}{(N-1)(N-2){S_y}^3} \quad \ldots\ldots\ldots\ldots \quad (3.3)$$

The observed data will be changed to point Rainfall using different statistical distributions methods. The most commonly distributions used to fit Extreme Rainfall Events are Log-normal distribution Type II, and Log Pearson Type III.

A) Lognormal distribution(Type II)

XT=10^YT

Where K_T = from the table of the variables and their means and standard deviation.

Y_T = ym+kT*S (Cs=0)

B) Log-parsec Type III

Y=log x is computed as:

KT=Z, for Cs=0, for Cs

KT=2+ (z^2-1) k+1/3(z^3-6z) k^2-(z^2-1) k^3+zk^4+1/3k^5

Where K = Cs/6

Y_T=ym+kT*Sy ,

XT= (10) ^Y_T

C) Gumble's Méthod

XT=Ym+kT*Sy

Where K_T= (YT-Yn)/Sn

Where Yn= mean and $Y_T = -\ln\left(ln\frac{T}{T-1}\right)$

Sn= standard deviation.

Using the D-Index test method determine the best statistical distribution to estimate the peak Rainfall event

$$\text{D-index}=\left[{}^{1}/{}_{X_m}\right] * \sum_i^6 abs\left(R - R^{"}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.4)$$

Using Flow magnitude estimation method identifying Irrigable area. The river base flow estimation is determined Using local information, Site observation top map, during field study

$$\text{Maximum irrigable area} = \frac{base\ flow\ (l/s))/)}{(Maximum\ demand\ (l/s/ha)} \ldots\ldots\ldots\ldots\ldots\ldots (3.5)$$

Using Complex Unit Hydrograph Methods (SCS).Design peak flood analyzed by Maximum probable flood is a hypothetical flood at a selected location, whose magnitude is such that there is no chance to exceed. It is estimated by combining the most hydrological and meteorological conditions considered reasonably possible at the particular location under consideration,

Estimate peak discharge from the given rainfall data and design flood analysis. This is widely adopted and more reliable method for flood estimation.

Using Peak flood analysis by complex hydrograph method is involves the preparation of standard unit hydrographs caused by rainfalls of specified durations.

Seepage head should be cheeked designing the impervious floor using Khoslas theories

The tail water level is used for deciding the bottom elevation of the downstream floor and to know where the hydraulic jump is formed.

$$\text{Average height } (\mathbf{H_{avg}}) = \frac{2*A}{L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.6)$$

The weir height is determined based on the maximum command area elevation which is required to irrigate the maximum possible irrigable area and consists of head losses: Across the head regulator, due to slope of main canal required to drive the full supply level in the main canal. Using the above methods design the proposed wire on Asher River

4. RESULTS AND DISCUSSIONS

4.1. Data Consistency Test

The daily heaviest Rainfall data of Dangela Metrological station from 1997 to 2007 is taken for the design. These data, which are not fully recorded, are abandoned and only these data, which are fully recorded, are taken for computation. Hence 11 years of daily heaviest rainfall data is available. These data should be checked for its consistency by outlier test. First the reliability of the data must be check

A) Checking Data Reliability

$$\delta_n / X * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4.1)}$$

Number of data = 11

Standard deviation, δ_{n-1} = 19.26

Mean, X=61.98mm

Standard error of mean, $\delta_n = \frac{\delta_{n-1}}{\sqrt{n}}$ = 5.81

Relative standard, $\delta_n / X * 100$ = (5.81/61.98)*100 = 9.37 %< 10%

Hence the data series could be regarded as reliable and adequate.

B) Data Outlier Test

This is done to check whether the adopted data is within the limited range or not.
Input data:-

$$C_s = \frac{N\sum(Y_i - \bar{Y})^3}{(N-1)(N-2)S_y^{\,3}} \ldots\ldots\ldots\ldots\ldots\ldots \text{(4.2)}$$

$$S_y = \sqrt{\sum \frac{\left(Y_i - \bar{Y}\right)^2}{(N-1)}} = \sqrt{\frac{0.16}{10}} = 0.1265 \ldots\ldots\ldots\ldots\ldots\ldots \text{(4.3)}$$

$$\sum RF = 681.8mm$$

$$\sum Y = 19.52mm$$

$$\bar{Y} = \frac{\sum Y}{N} = \frac{19.52}{11} = 1.77$$

$$\sum (Y - \ddot{Y})^2 = 0.16$$

$$C_S = 0.604$$

C) Tests for Outliers

Outliers are data points that depart from the trend of the data. The remaining detention or retention of these outliers can significantly affect the magnitude. As shown from the above calculation the station Skew is greater than 0.4, test for high outlier is considered first.

Test for higher outlier

Higher outlier

$$Y_h = \bar{Y} + K_n S_y \qquad (4.4)$$

Where: $\bar{Y}$ = mean of data in log unity

K_n = From table for sample size

N = Sample size

C_s = Skew ness coefficients

From Table for N=11 $y = 1.77$, $S_y = 0.126$, $K_n = 2.08$

Skew ness coefficients C_s =0.604

Higher outlier $Y_h = \bar{Y} + K_n S_y = 1.77 + 2.088 * 0.1265 = 2.034$

$Higher outlier = (10)^{2.034} = 110.05mm$

The highest recorded value is (103.5mm) is less than high outlier (110.05mm). Therefore, there is no higher outlier.

Test for lower outlier

Lower outlier $Y_l = \bar{Y} - K_n S_y$ ………………………………………………………… (4.5)

$Y_h = \bar{Y} - K_n S_y = 1.77 - 2.088 * 0.1265 = 1.5058$

$$\text{lower outlier} = 10^{1.50} = 32\text{mm}$$

The Lowest recorded value is (37.3mm) which is greater than lower outlier (32mm). Hence, no lower outlier. Therefore, the recorded data is consistent for both outliers and it is possible to use it for analysis

4.2. Selection of Distribution

The most commonly distributions used to fit Extreme Rainfall Events are Log-normal distribution Type II, and Log Pearson Type III. The results of the analysis are shown in the following table.

Table 2: Peak Rainfall estimates using different statistical distributions

Probability	K	Frequency factor	Frequency factor, KT	Standard normal variance, Z	Normal variance	X_{50}	Remark
P=1/T	K=Skewcoe ff./6	w=(ln(1/P^^{2}))^$^{1/2}$					
0.02		2.7971	2.054	0		101.56	Normal
0.02		2.7971	2.592			111.92	Gumbel
0.02		2.7971	2.054	2.037		108.96	Log Normal Distribution
0.02	**0.073**	**2.7971**	**2.054**	**2.281**	**2.066**	**116.49**	**Log-Person Type.III**
0.02	0.073	2.7971	2.054	2.281		105.93	Pirson Type IIIl

The D-Index test is believed to be the better goodness to fitness test in many literatures. Hence in this study it was used to determine the best statistical distribution to estimate the peak rainfall.

The D-index for the comparison of the fit of various distributions in upper tail is given as

$$\text{D-index}=\left[{}^{1}/_{X_m}\right] * \sum_i^6 abs\left(R - R^{"}\right)\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.7)$$

The smallest D index value was found to be for the Log Pearson Type III distribution, which is 0.226. Accordingly, the design rainfall was found to be 116.49mm for the Log Pearson Type III distribution, from (Table 2). United States water resources council (USWRC) guidelines for determining flood flow frequency. Here xi and xi' are the i^{th} highest observed and computed values for the distribution. The distribution giving the least D-index is considered to be the best-fit distribution

Table 3: D-index test

Rank	R	Normal	Gumbel EVI	Log Normal	Log person Type III	Person Type III
		R -R''	R –R"	R -R''	R -R''	R -R''
1	104	10.819	7.38	8.358	5.465	8.859
2	84.4	1.986	3.371	3.074	3.075	1.988
3	74.3	1.7	1.28	0.566	1.452	0.909
4	66.9	3.964	0.383	1.268	0.041	2.776
5	60	6.303	2.631	3.578	2.262	4.933
6	60	1.982	1.436	0.484	1.739	0.587
Sum(X_m)		26.754	16.48	17.328	14.034	20.051
D-index		0.432	0.266	0.28	**0.226**	0.323

4.3. Base Flow Estimation

In Asher River, the discharge magnitude was estimated to be around 240l/sec in December during dry season to plan the size of the irrigable area that the project supports considering the water potential available, downstream utilization allowance and crop water requirement. The maximum quantity of water that the crop requires with 50% efficiency and 18hr irrigation may be 1.71l/s/ha.

The size of irrigable area is determined as fallow.

$$\text{Max irrigable area} = \frac{\boldsymbol{base\ flow\ (l/(s)}}{\mathbf{max}\ \boldsymbol{demad(\ l/(s/(h\_a)}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.6)$$

Where = Maximum demand =1.71l/sec/ha

Base flow=240m/sec

Maximum irrigable area 140 hectare

4.4. Design Flood Analysis

Have described earlier 11 years daily heaviest Rainfall data obtained from Dangila Meteorological station is used for determination of maximum probable flood. Maximum Probable Flood (MPF) is a hypothetical flood at a selected location, whose magnitude is such that there is no chance of its being exceeded. It is estimated by combining the most hydrological and meteorological conditions considered reasonably possible at the particular location under consideration.

4.4.1. Design Storm Analysis

From the observed data point rainfall was designed using different statistical distributions. From the above Table Log Pearson Type III distribution has higher rain fall depth value of 116.49mm is selected for our analysis to minimize the risk. So the point design rain fall is 116.49mm

4.4.2. Time of concentration

The time of concentration is calculated using the Kirchf-Formula

$$T_c = \sum 0.948 \left(\frac{Li^3}{Hi}\right)^{0.38}$$

Where: - Tc = Time of concentration (hrs)

Li = Maximum length of flow (kms)

Hi = Difference in elevation between the outlet and the remote point (m)

Considering the non-uniform slope of the stream, the stream divides in to three reaches so as to increase the accuracy of the calculation of the time of concentration and the calculation is shown in the table below

Table 4: Estimation of time of concentration

S. No	Stream length	Distance	Elevation	Elevation Difference	T_C	Remark
1	0	0.00	1862.00	0	0	Weir axis
2	23	23.00	2365.00	503.00	3.23	
3	43	20.00	2869.00	504.00	2.74	Remote point
sum		43.00		1007.00	5.98	T_C
sum		43.00		1007.00	5.98	T_C

The time of concentration computed by dividing the longitudinal profile of the river into segments is nearly 5.98hr. Time of concentration computed by considering the longitudinal profile of the river as one segment is nearly 6.30hrs. Let us take the time of concentration as the average of the two cases Time of concentration,

$$Tc_{avg} = \frac{5.98 + 6.30}{2} = 5.75$$

4.4.3. Design Rainfall Arrangement

A. Areal Rainfall

As the area of the catchments gets larger, coincidence of all hydrological incidences becomes less and less. This can be optimized by changing the designed point rainfall to areal rainfall. The conversion factor is taken from standard table that relate directly with the size of watershed area and type of the gauging station.

For the case of Asher irrigation project,

Total watershed area = 58.6 Km^2

Type of gauging station = Daily rainfall (24 hr.)

Conversion factor = 92.7 %

Arial Rainfall = (Point Rainfall) x (Conversion factor)

Arial Rainfall =116.48x92.7/100=108mm

B. Rainfall Profile

Rainfall profile is the distribution of design rainfall with respect to time in the whole watershed area. It needs developed models for the selected drainage area. But there is sufficient modeling data in the vicinity and adaptation of standard curves is the only option. Designers of this project have adopted the standard curve from IDD Manual and used to compute rainfall profile of the project area.

Table 5: Design Rainfall Arrangement

Time (hr)	Design areal Rainfall (mm)	Rainfall Profile (%)	Rainfall Profile (mm)	Incremental Rainfall (mm)	Ascending Order	Rearranged order	Rearranged R.F.incr. (mm)	Rearranged R.F cumulated (mm)
0-1	108	35	37.8	37.8	1	6	3.24	3.24
1.0-2.0		48	51.84	14.04	2	4	8.64	11.88
2.0-3.0		58	62.64	10.8	3	3	10.8	22.68
3.0-4.0		66	71.28	8.64	4	1	37.8	60.48
4.0-5.0		72	77.76	6.48	5	2	14.04	74.52
5.0-6.0		75	81	3.24	6	5	6.48	81
6.0-12.0		90	97.2	16.2				
12.0-24.0		100	108	10.8				

c. Runoff Analysis

Input data

- Design Point Rainfall = 116.48mm
- Area rain fall conversion factor = 92.7%
- Curve number at antecedent moisture condition III = 92.71
- Catchments Area, A = 58.6 Km^2
- Tc=3.5 hr, D = 1hr., Tp =2.6hr; Tb =7hr; Tr = 10.8hr.

  Direct run-off, $Q = (I-0.2*S)^2/(I+0.8*S)$

Where, I=Rearranged cumulative run-off depth (mm

S=Maximum run of potential difference, =(25400/CN)-254

Peak run-off for incremental;

Qp =0. 21*A*Q/Tp

Where, A=Catchment area= Km^2

T p=Time to peak hr.

Q = Incremental run-off (mm)

Table 6: Direct Runoff Computation

Time (Hr)	Incremental Rain fall (mm)	Accumulative R.F (mm)	Direct Run off		
			Accumulative (mm)	Incremental (mm)	Incremental Loss (mm)
0-1.0	3.24	3.24	0	0	3.24
1.0-2.0	8.64	11.88	2.23	2.23	6.41
2.0-3.0	10.8	22.68	9.03	6.8	4
3.0-4.0	37.8	60.48	41.73	32.7	5.1
4.0-5.0	14.04	74.52	54.96	13.23	0.81
5.0-6.0	6.48	81	61.15	6.19	0.29

Table 7: Computation of peak discharge for each incremental runoff

Time (hr)	Incremental Run off	qp for 1mm incremental run off (m3/sec)	qp for incremental run off (m3/sec)	Incremental Hydrograph		
				Begin time	Peak time	End time
0-1	0	4.73	0	0	2.6	7
1.0-2.0	2.23	4.73	10.57	1	3.6	8
2.0-3.0	6.8	4.73	32.19	2	4.6	9
3.0-4.0	32.7	4.73	154.77	3	5.6	10
4.0-5.0	13.23	4.73	62.62	4	6.6	11
5.0-6.0	6.19	4.73	29.28	5	7.6	12

Table 8: Peak discharge analysis using complex hydrograph method

Time	H1	H2	H3	H4	H5	HT	Remark
0	0					0	
1	0					0	
2	4.07	0				4.07	
2.6	6.5	7.43	0			13.93	
3	8.13	12.38	0	0		20.51	
3.6	10.57	19.81	35.72	0		66.1	
4	9.61	24.76	59.53	0		93.9	
4.6	8.17	32.19	95.24	14.45		150.05	
5	7.21	29.26	119.05	24.08	0	179.61	
5.6	5.77	24.87	154.77	38.54	6.76	**230.7**	**Q PEAK**
6	4.8	21.95	140.7	48.17	11.26	226.88	
6.6	3.36	17.56	119.6	62.62	18.02	221.15	
7	2.4	14.63	105.53	56.93	22.52	202.01	
7.6	0.96	10.24	84.42	48.39	29.28	173.29	
8	0	7.32	70.35	42.7	26.62	146.98	
8.6		2.93	49.25	34.16	22.63	108.95	
9		0	35.18	28.46	19.96	83.6	
9.5			17.59	21.35	16.64	55.57	
10			0	14.23	13.31	27.54	
10.5				7.12	9.98	17.1	
11				0	6.65	6.65	
11.5					3.33	3.33	
12					0	0	

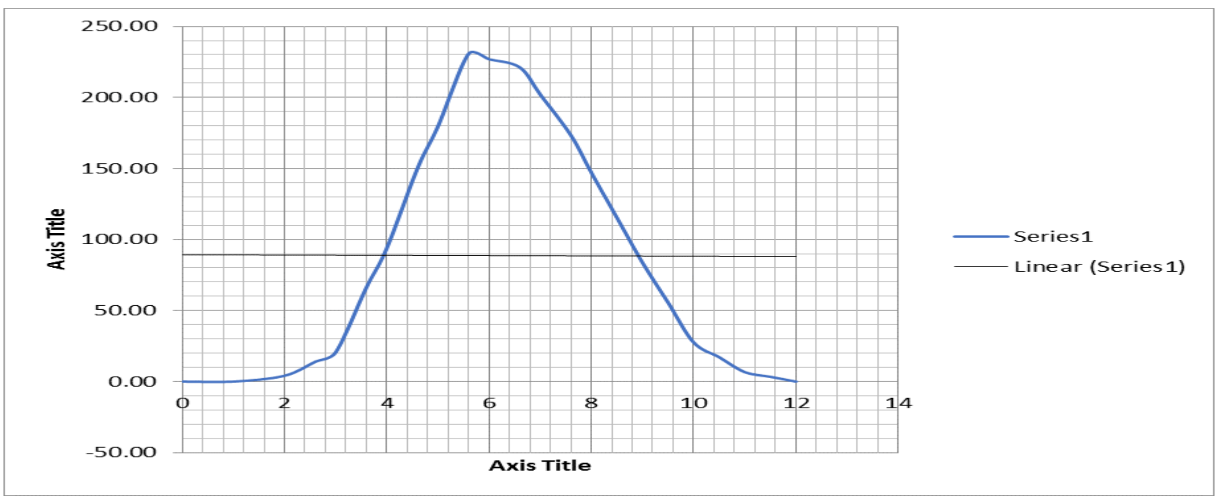


Figure 6: Hydrograph curves

From the Figure 6 analysis, the 50 year return period design runoff is **230.7m**^{3}/sec

4.4.4. Tail Water Depth Computation

Tail water depth of the river is equal to the flood depth and amount at the proposed weir site before construction of the weir. It is used to realize the flood feature after the hydraulic jump. During field visit, the flood mark of the river at the proposed diversion site was marked based on resident's information and physical indicative marks. The river cross-section was surveyed. The water discharge is calculated by Manning's open channel formula. Basic inputs for the analysis and the detail procedure are described as follows

I. Tail Water Depth

Average bed slope estimation

From table 9 below calculate the average slope of the river by the following formula.

A= $\frac{H_n+H_{n-1}}{2} * L$,$H_n$ elevation at n^{th} station

Table 9: Elevation along the river cross-section

SR NO	partials(L)	cumulative des	elevation	Cumulative height(H)	Area(m^2)
1	0	0.00	1937.68	0	0
2	51.11	51.11	1937.37	0.31	7.92205
3	28.80	79.91	1936.43	1.25	22.464
4	43.77	123.68	1936.79	0.89	46.8339
5	11.77	135.45	1936.19	1.49	14.0063
6	2.82	138.27	1936.01	1.76	4.4477
7	5.70	143.96	1935.65	2.03	10.53575
8	7.14	151.10	1936.13	1.55	12.7806
9	5.75	156.85	1936.38	1.3	8.19375
10	39.15	196.00	1936.00	1.68	58.3335
11	2.5	198.5	1935.75	1.93	4.5125
sum	**198.5**				**190.0301**

The total length =198.50m and total cross sectional Area=190.301m^2

Average height ($\mathbf{H_{avg}}$) =1.91m

Average slope, $(S_{avg}) = \frac{Havg}{l} = \frac{1.91}{198.5} = 0.009646 m/m = 0.9646\% \cong 1\% = 0.01$

Also determine the bed slope ($S_{avg.}$) by plotting the river profile as follows:

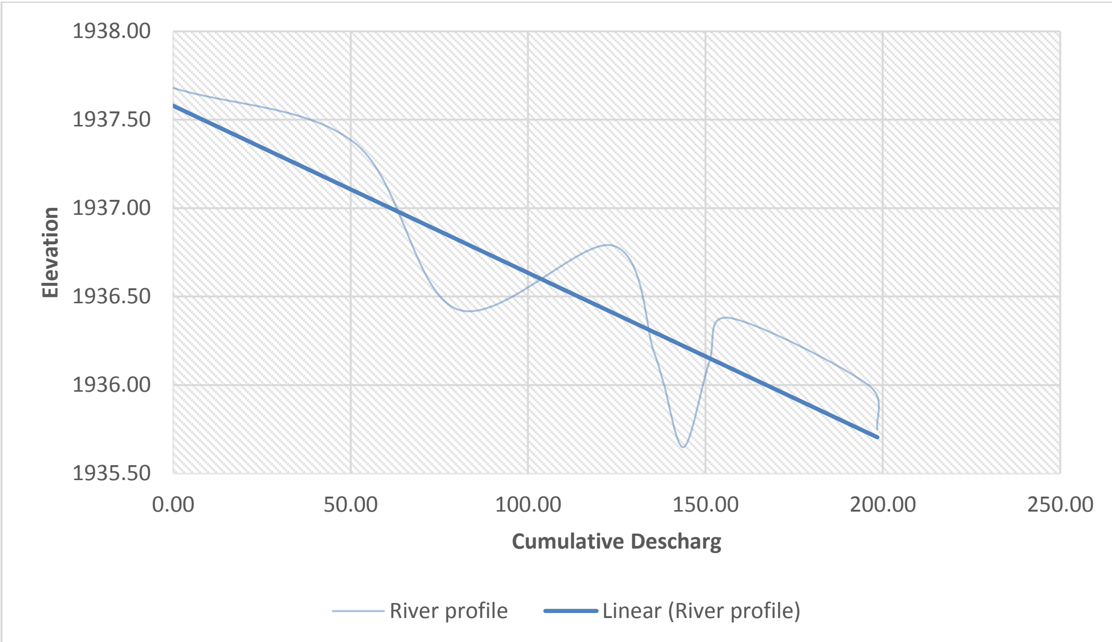


Figure 7: River profile computation

II. Manning's Roughness coefficient

The Manning's roughness coefficient is taken from standard table based on the river nature. The river at the headwork site has got braded feature and curving nature. The riverbanks are defined and relatively smooth. Manning's roughness coefficient (n = 0.035) is adopted.

III. River Discharge

Manning's roughness coefficient, n =0.035

Average river bed slope, S = 0.0079

$$V = \frac{1}{n} \times R^{2/3} \times \sqrt{S}\,, \quad \text{..(4.9)}$$

Where, R = Hydraulic radius = (Area/Perimeter)

Roughness coefficient, N=0.035

Average River bed slope=0.0079

Table 10: River discharge computation at different stage of flow

Elevation	Depth(d)	Area(m^)	Perimeter(p)	Radius(m)	Velocity(m/se)	discharge Q(m^3/se)
1938.25	0	0.3	4.82	0.062241	0.426	0.12771
1938.75	0.5	5	14.12	0.354108	1.357	6.783389
1939.25	1	14.35	30.13	0.4762695	1.653	23.721399
1939.75	1.5	32.85	44.7	0.734899	2.207	72.511543
1940.25	2	48.29	46.37	1.041406	2.785	134.47983
1940.75	2.5	63.47	47.24	1.343565	3.3	209.47271
1941.25	**3**	76.95	48.04	1.62457	3.712	**230.683**
1941.75	3.5	116	81.78	1.418440	3.422	396.93414

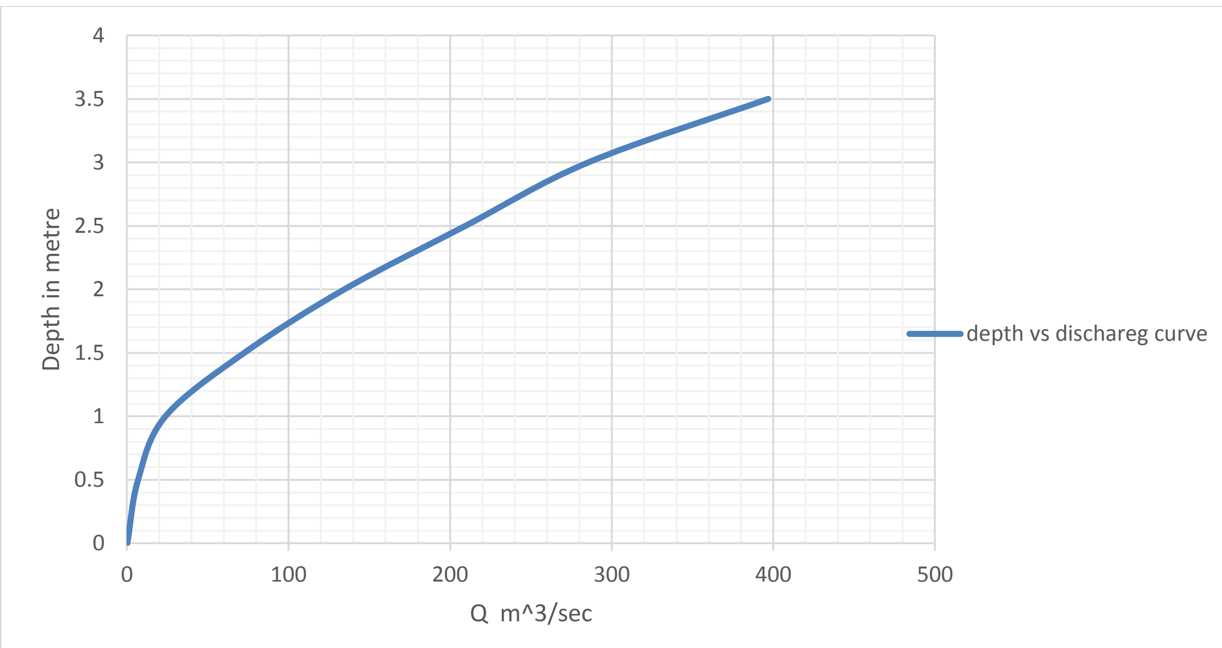


Figure 8: Depth vs. Discharge curve

From The Figure 4 Below (Depth- Discharge Curve) And *Table 10: Tail Water Depth Estimation* The Tail Water Depth Equivalent To The Flood Discharge (Q =230.7 m^3/Sec) Is Found About 3m.

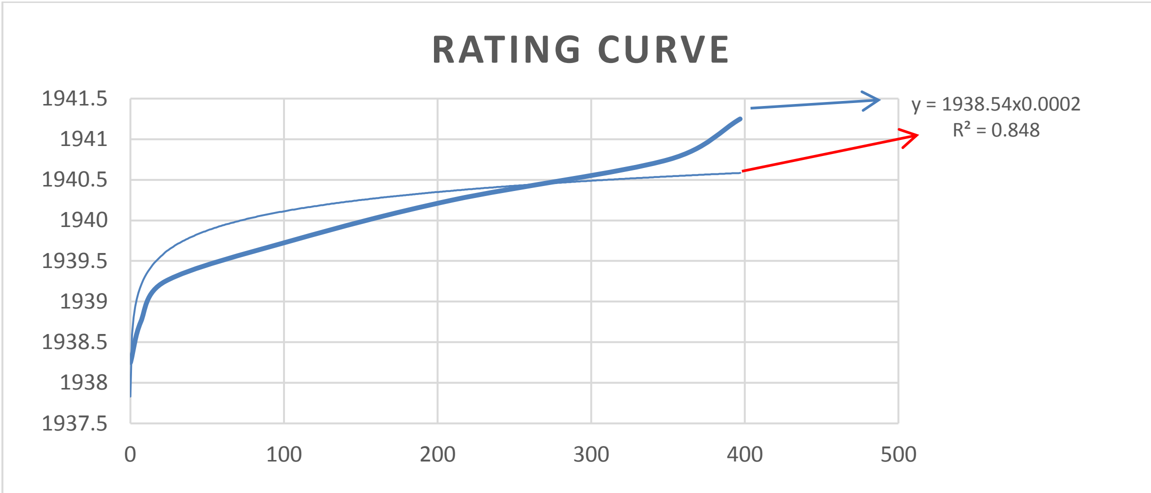


Figure 9: Rating curve computations

4.5. Headwork Site Selection

Having decided upon the location of weir the actual site is selected with the following considerations. A reasonably wide and well defined channel with reliable banks is favored, The associated canal alignment should enable adequate command without excessive excavation or embankment, With respect to the adjoining land surface, the elevation of water surface upstream of the weir should not be so low as to require an excessively high weir to divert the water at the intake, Easy arrangement of flow diversion during construction and availability of construction material at the nearest place.

4.6. Weir Type Selection and Parameters

(i) Weir Type Selection

When in selection of the weir type, it should have to consider the availability of construction materials, simplicity of the structure/practicality, nature of foundation and the river bed material as well as weir height. The peak discharge estimated is 230.7m^3/s. A weir type that can dissipate the energy of water falling from height needs to have better energy dissipation efficiency, because the weir shape is capable of resisting the impact from a jet or pressurized stream of fluid of water. In addition to this the river carries sizable boulders and cobbles towards the diversion site during flood season. In this respect an ogee type weir is preferable. Besides constructing a broad crested type weir in a river reach where the risk of boulder in crushing its sharp edge is not advisable. Hence selecting an ogee weir is advisable to reduce the impact of this boulder on provided structure. The proposed weir is to be constructed by cyclopean concrete with reinforced concrete capping. The cyclopean concrete consists of 60% C-25 concrete and 40% graded stone of size less 10cmm diameter, with external cover of single reinforced C-20 concrete

(ii) Geological Characteristic

The project is surrounded by hill, some undulated, flat and mountain lands, which covered by clayey silt, with insufficient rock fragments, reddish brown dry, stiff to very stiff residual soils and basaltic rocks.

(iii) U/s and D/s of the Head Work

Presently the majority of the bed is covered by surface flowing water. The surface sediments are dominated with gravels, cobbles and boulders, silt etc. They are rounded to sub-rounded, strong, and dominated with basaltic rock

(iv) Nature of the Foundation Right and left Abutment

At the proposed weir site, the stream has relatively steep to vertical slope of about 3 to 4m height. The surface bank is completely made up of soil classified as floodplain deposit of silt clay texture. Presently the bank is not stable. The left bank has relatively moderate to gentle slope, with height of about 1.7m to 2m. It is made up of similar geologic materials to the right bank, but has different depth or thickness. The top is the same dark-brown colored salty-clay soil. Therefore the retaining wall is provided to the d/s and u/s part of the weir for both right bank and left bank

(v) Availability of Construction Materials

During this field work a required natural construction materials have been assessed. Here source areas for rock, clay borrow areas and fine aggregate, sand have been indicated.

4.7. Weir Height

The weir height is determined based on the maximum command area elevation which is required to irrigate the maximum possible irrigable area and consists of head losses: Across the head regulator, due to slope of main canal required to drive the full supply level in the main canal.

The analysis is shown as follow:

- River bed level=1938.25m.a.s.l.
- Maximum command area elevation= 1939.25m.a.s.l
- Maximum Flood elevation=1941.66m.a.s.l.
- Distance from the head work site=670m
- Main Canal slope=1:1000
- Head loss due to slope=0.67
- Head regulator loss=0.1
- Canal flow depth =0.4m
- Free board=0.2
- Canal outlet level=1939.25m+0.67m+0.1m =1940.02masl
- Weir crust level= 1939.25masl+0.67m+0.1m+0.4m=1940.42masl
- weir height=1940.42-1938.25masl=2.17m

Accordingly the weir height fixed to be 2.17m and the corresponding crest level fixed to be 1940.42m.a.s.l
From the above table (depth- discharge table) and figure the tail water depth equivalent to the flood discharge 230.7m^3/s is found about 3m.Tail water level, (maximum D/s HFL) = 1938.25m a.m.l + 3m = 1941.25m.

4.8. Weir Crest Length

Length of the weir depends on the physical feature of the river at the site of the weir and taking into account the area of submergence on upstream side of weir axis. From the Lacey's regime width formula, the width of the river and geology of the abatements at the proposed weir axis, the crest length of the over flow weir section can be determined.

Lacey's regime width=4.75(Q) ^0.5=72.15m, this is too large

Actual river crest length is equal to bank to bank width of the over flow section of the river from the given top map is =25m

Considering the actual site conditions of the river banks stability and width of the river channel, the crest length of the weir is considered as 25m.

4.9. Hydraulics of the Weir

4.9.1. Discharge Over the Weir Section

The over flow over the weir would be affected by the shape of the weir, because of the coefficient of flow discharge Cd varies with the type of weir and shape. Since the type of weir selected for Asher project is ogee and the coefficient of discharge C_d was assumed to be 2.2.The design discharge ($Q_{d)}$ formula for ogee type weir is

$$Qd = Cd * L * He3/2 \qquad (4.9)$$

Take the e following

- G=Specific gravity of floor material =2.3
- H=height of the weir=2.17m.
- Q= designed discharge =230.7m^3/s
- The total weir crest length (L) =25m
- Coefficient the discharge c=1.7

Head over the weir crest (He) = maximum depth of water over the crest (over flow depth + approach velocity head) or he Head over the weir crest,

$$He = \left(\frac{Q}{1.7 \times L}\right)^{2/3} \qquad (4.8)$$

$He = 2.6m$ Add over the weir crest level.

River bed level = *1938.25m* asl

Weir top level =1938.25+2.17=1940.42m asl

U/s Total energy level = 1940.42 + He = 1940.42+2.6 m= 1943.02m asl

Maximum flood elevation before constriction =D/s HFL= 1938.25+3= 1941.25m a s l

High flood level before construction of the weir d/S HFL 1938.25+3m=1941.25m. From tail water curve

Therefore, the effect of the weir at peak flow condition is negligible.

U/s HFL = U/s bed level+ weir height + Hd ………………………… (4.9)

Hd =depth of water over the weir crest & calculated by using ogee weir principle.

The discharge coefficient for this ogee weir is affected by depth of approach and d/s apron interference and submergence effect. Hence it is fixed as shown below.

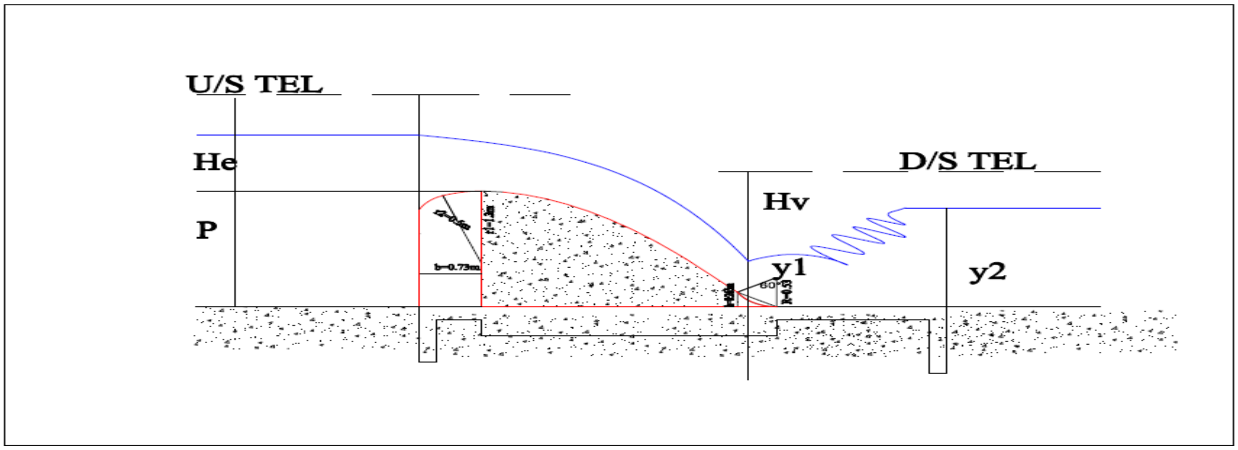


Figure 10: weir profile

I. **Based on depth approach**

Q=CLHe^3/2   C=2.2

He= (Q/CL) ^2/3

He=2.6   take as He= 2.6m

The velocity approach, Va is calculated as follow as.

- $V = Q/(L * (p + Hd))$
- $Hd = He - V^2/(2 * 9.81)$
- $V^2/(2 * 9.81) = 2.12 - Hd$
- $Q^2/(((25 * (0.50 + Hd))^2 * (2 * 9.81)) = 2.12 - Hd$

Hd	Q^2/(((25*(0.5+Hd))^2*(2*9.81))	2.12-Hd
1.72	0.88	0.88

$Ha = \text{Velocity head} = He - Hd = 2.12 - 1.56 = 0.56, \quad Va = (ha * 2g)^{0.5} = 4.155$

$\text{Weir height}, h = 2.1\ m,\ Hd = 1.72\ m,\ He/Hd = 2.6/1.72 = 1.5,$

$$h/Hd = 2.1/1.72 = 1.221$$

For h/Hd=1.2 and He/Hd=1.5,C/Cd=1 from the graph, the discharge coefficient.

Thus, C=2.2 is correct based on approach depth condition. 1940.42m

II. Based on D/S Apron &Submergence effect

$$\frac{U}{s}HFL \ = \frac{U}{s}bed\ level + \ weir\ height + \ Hd \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.10)$$

Hence U/s HFL=1938+2.17+1.72=1942.14masl

U/s TEL=U/s HFL +approach velocity head=1942.14+0.88=1943.02m

Tail water depth=3 m as computed previously.

h_d = U/S TEL-D/S HFL=1943.02-1941.25=1.77

(h_d+ d)/He= (1.77+3.41)/2.6=1.99

Hence 1.99 >1.7, the d/s apron interference for C is negligible effect.

U/S TEL=U/s HFL+ ha=1942.14+0.88=1943.02m

D/S TEL=D/s HFL+V2/2g=1941.25+ (3.11^2/2*9.81)=1941.25+0.493=1941.74m

(The velocity head in this case is computed using manning eqn. From table 2-11)

Afflux=U/s TEL-D/S TEL

Afflux =1943.02m-1941.74m=1.28m

4.9.2. Design of the Weir Profile

Based on experiments approval to avoid negative pressure including consideration of practicability, hydraulic efficiency, stability and economy, the equation is derived.

Vertical U/S face weir X1.85=2*$He^{0.85}$*y

Hence from the construction point of view and stability, it is better to provide 1:1 D/S slope.

$Y=X^{1.85}/ (2*He^{0.85})$ ……………………………………………………… (4.10)

Where, He=H_d+ ha

- $Y=X^{1.85}/ (2*He^{0.85})$
- Y= $X^{1.85}$ /2*(1.72+0.493) ^0.85
- $Y=X^{1.85}/3.93$

To have efficient curvature, it is better to determine the tangent point.

$$dy/dx = v/h = \ dy/dx = 1/1$$

$$y/dx = 1.85 * x(1.85 - 1)/3.93 = 1/1$$

$$x0.85 = 3.93/1.85$$

For X = 2.43m and Y=1.36m

Based on this value, the coordinate of x=0 to x=2.43 is tabulated as follow

Table 11: Coordinates of the downstream weir profile with respect to the weir crest level

X	**0**	**0.25**	**0.5**	**0.75**	**1**	**1.25**	**1.5**	**1.75**	**2**	**2.25**	**2.32**	**2.43**
Y	**0**	**0.02**	**0.07**	**0.16**	**0.26**	**0.40**	**0.56**	**0.74**	**0.95**	**1.18**	**1.25**	**1.36**

The u/s profile from the axis is computed using the following eqn.

y=0.724*(x+0.27*He) ^1.85/He^0.85+0.126*He-0.4315*He^0.375*(x+0.27He) ^0.625

y=0.724*(x+0.7) ^1.85/2.25+0.3276-0.617*(x+0.7) ^0.625

This curve should evaluated up to x=-0.27*H He=-0.27*2.6=-0.7

Table 12*:* Upstream curve profile values

X	0	-0.1	-0.2	-0.3	-0.4	-0.5	-0.6	-0.7
Y	0	0	0.02	0.04	0.07	0.12	0.19	0.33

Table 13: Upstream Curve Parameter and Values

U/s Face curve		
Parameter	Relation	Value
He	2.6	2.6
r1	0.5*He	1.3
r2	0.2*He	0.52
A	0.175*He	0.455
B	0.282*He	0.73

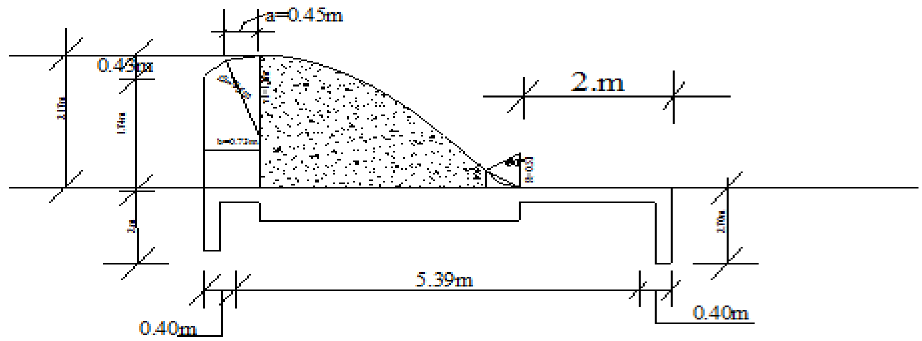


Figure 11: Weir Cross-Section and dimension

4.9.3. Hydraulic Jump Computation

Conjugate depth and jump occurring level of the flow has been computed for different discharges. And finally the tail water and post jump depths versus discharge curve has been plotted. Input data for the analysis:

Peak discharge, Q = 230.7 m3/s

Unit Discharge, q= Q/25 = 9.2 m^3/s/m

Head over the weir, He = 2.6m

Weir crest level = 1938.25+2.17=1940.42m asl

U/S TEL = 1943.02m and D/S TEL=1941.74m

Head over the weir crest = TEL – Crest level = 1943.02m-1940.42m =2.6m

Using energy equation, neglecting losses between **A** and **B,** and considering similar datum, as shown in the above figure above, the analysis is done as under. I.e. Head (A) =Head (B)

y1	Y1^3-5.1*y1^2+4.31=0	Remark
1.029	0.00	Using iteration

Hence using iteration, the value of y1=1.029 m

The value of y2 is evaluated using the following equation

- $F_1^2 = q2/gy1^3$ ……………………………………………(4.12)

  Where F1=Fraud number

$$Fr = \frac{V1}{\sqrt{g \times Y1}} = 2.82$$

- $Fraud\ Number,$

$$Fr\text{^}2 = 9.2^2/(9.81 * 1.029^3) \ = 7.967\ using\ eqn. - - - - - - - - - - (1)$$

$$Y2=\frac{Y1}{2}*((8*f1^2+1)^{0.5}-1)----------------\qquad(2)$$

$$Y2=\left(\frac{1.029}{2}\right)*((8*7.967+1)^{0.5}-1)=3.62\ m\ using\ eqn\ \ldots\ldots\ldots..(2)$$

Cistern level=D/s HFL-Y_{2}-

CL=Cistern level= 1941.66-3.62m=1938.04 m, this is greater than 1940.02m

Hence take the cistern level =1938.04 m

Hydraulics jump calculation and cistern length

Weir crest length = 25m

Weir height (h) = 2.17m

Pre-jump depth = y1

Post -jump depth =y2

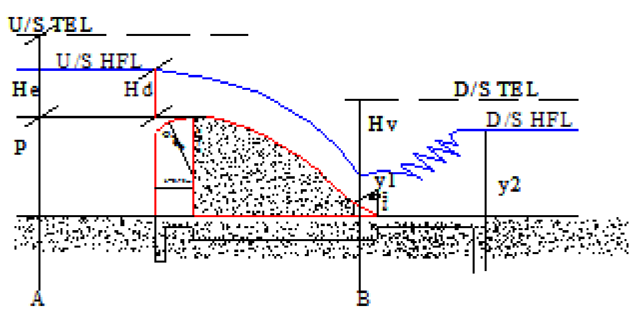


Figure 12: Hydraulic Jump Profile

Neglecting losses between point A and B and considering similar datum using the principle of energy equation .refer the weir profile fig .7

h+ He = y1 + ha where He = 2.03m

$$q=\frac{Q}{L}=\frac{230.7m3/s}{25m}=9.23m2/s$$

head loss due to approach velocity$(ha)=\frac{q2}{2*g*y2}=\frac{9.23^2}{2*9.81*{y_1}^2}$................. (4.13)

Therefore 2.17m+2.0264m = y1+2.053/y_1^2 q^2/ (2g*y1^2) +Y1 =He+h

Table 14: Iteration for pre jump depth calculation using a table

q^2	Y_1	Y_2	H_e+h	$\frac{q^2}{2*9}$	Y_1 ^2	$\frac{q^2}{2*g*{y_1}^2}+y_1$
40.274	0.912	2.579	4.2	2.053	0.831	3.38

Thus: After iterations pre jump (Y1) =0.91m

V1=q/y1=6.346/0.912=6.96

$Fr = \frac{v1}{\sqrt{gy1}} = \frac{6.96}{\sqrt{9.81*0.912}} = 2.326$.

The F1 is in between 1.7 and 2.5, the type of jump formed is called weak jump.

$$Y_2 = \frac{Y_1}{2}\left(\sqrt{1+8*{F_r}^2}-1\right) Y_2 = \frac{0.912}{2}\left(\sqrt{1+8*2.326^2}-1\right) = 2.579m$$

Cistern length (L) = 5*y_2=5*2.326=11.996m$\cong 12m$

4.9.4. Design of Energy Dissipation

U/s and d/s cutoff

Discharge (Q) =230.7m^3/sec

Weir crest length (L) =25m

Intensity of discharge =Q/L =230.7/25=9.22m^3/se/m

Silt factor f =1.76$\sqrt{\boldsymbol{mr}}$=1.76$\sqrt{\mathbf{1}}$ =1.76

Scour depth(R) = 1.35(q^2/f) ^ (1/3) =1.35(9.22^2/1.76=3.64m

U/S HFL =1942.14m.a.s.l

River bed level=1938.25m.a.s.l

D/s HFL before retrogression = 1941.25m.a.s.l U/s cut off

U/s pile level = u/s HFL-1.5R = 1942.14-1.5*3.64 =1936.68m.a.s.l

Depth of u/s pile (d_1) =river bed level-U/s pile level =1938.25m.a.s.l-1936.68m.a.s.l=1.57m takes 1.6m

A) D/s cut off

D/s cutoff level = d/s HFL -1.5R = 1941.25-1.5*3.64=1935.79m.a.s.l

Depth of downstream pile (d_2) = river bed level –d/s cut off level = 1938.25-1935.79 =2.4

4.10. Impervious Floor

. It may occur under no flow condition where the head difference between the weirs crest level and the downstream bed level or under a full discharge condition with a hydraulic jump in the stilling basin.

Khosla's theory

Design of impervious floor thickness

Maximum seepage (H_s) =2.17m

Total creep length (L_d) =27m

Bottom width (B) =3.5m

At the toe

$$h = \frac{H_S*(\text{Length}-(2*\text{upstreamcutoffdepth}+B+\text{nominalupstramlength}))}{\text{Lenth}} \quad \text{.................} (4.14)$$

$$\boldsymbol{h} = \frac{2.17*\left(27-(2*2+3.5+1.5)\right)}{27} = 1.43$$

The thickness of the floor at toe (t) $=\frac{4}{3}*\frac{h}{G-1} = \frac{4}{3}*\frac{1.4331}{23-1}$ 1.47. Provide 1.5m

At 3.5 m away from the toe

$$h=\frac{H_S*(\text{total creep length except } u/s \text{ apron}-(2*u/s \text{ cut off}+B+\text{nominal apron length})}{\text{total creep length except } u/s \text{ apron}}$$

$$= \frac{2.17*(25-(2*2+3.5+1.5)}{25} = 0.87$$

∴ Thickness required (t) $=\frac{4}{3}*\frac{0.868}{2.3-1} = 0.89 \cong 1$

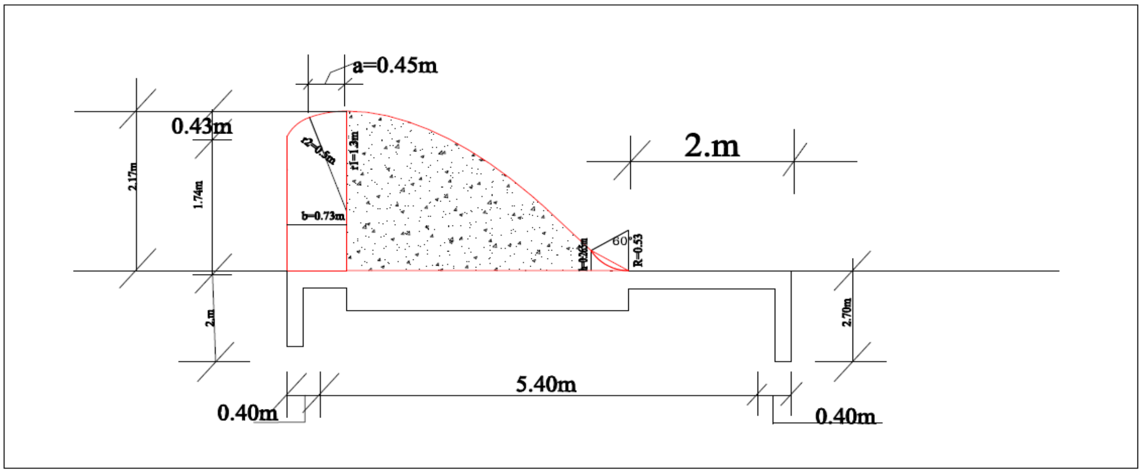


Figure 13: Apron Thickness of Weir Section

Pressure calculation at key points

Φ is take the water head percentage at key points.

- **U/s pile**

$\alpha = \frac{total\ floorlength(L)}{d_1} = \frac{17}{2}$=8.5

$\lambda = \frac{1+\sqrt{\alpha^2+1}}{2} = \frac{1+\sqrt{8.5^2}+1}{2}$=4.78

$\phi_D = \frac{100}{\pi} * COS^{-1}(\frac{\lambda-1}{\lambda} = \frac{100}{\pi} * COS^{-1}(\frac{4.78-1}{4.78})$=20.98%

$\phi_E = \frac{100}{\pi} * COS^{-1}(\frac{\lambda-2}{\lambda}) = \frac{100}{\pi} * COS^{-1}(\frac{4.78-2}{4.78})$ =30.26%

ϕ_{D1}=100-ϕ_D=100-20.98=79.02%

ϕ_{C1} =100-ϕ_E=100-30.26=69.74%

Correction for u/s pile

$\phi_{C1} = \left(\frac{\phi_{D1-}\phi_{C1}}{upstreamcutoffdepth}\right) * tu/s = \frac{79.02-69.74}{2} * 0.5$=2.32

Correction for mutual interferences in u/s pile

Total floor length (L) = 17m

Effective length (b') =total length –sum of pile thickness=17-0.4*2=16.2m

Depth (D) of d/s pile =d/s cut off depth –thickness of floor at 6m from toe

2.75-1.00= 1.75m

Depth of the u/s depth (d) =u/s cut off depth – normal thickness of U/S apron

2-0.5=1.5m

Correction(c) = (19*$\sqrt{D * b'}$)*$\frac{d+D}{b}$=19*$\sqrt{1.75 * 16.2}$ *($\frac{1.5+1.75}{17}$ =1.20. since the point C1 is in the rear in the direction of flow, the correction is positive.

Corrected ϕ_{c1}=69.74+6.96+1.2=73.25%

D/s pile

$$\alpha = \frac{total\ floof\ length}{dawn\ stream\ cut\ off\ depth} = \frac{17}{2.75} = 6.18$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 6.18^2}}{2} = 3.63$$

$\phi_{D2} = \frac{100}{\pi} * COS^{-1}(\frac{\lambda-1}{\lambda})$ =$\frac{100}{\pi} * \frac{3.63-1}{3.63}$ =24.22ϕ_{E2}=$\frac{100}{\pi} * COS^{-1}(\frac{\lambda-2}{\lambda})$ $\frac{100}{\pi} * \frac{3.63-2}{3.63}$= 35.20%

Correction for floor thickness (ϕ_{E2})

$\frac{\phi_{E2}-\phi_{D2}}{d_2} * t\ at\ teo$= $\frac{35.20-24.22}{2.75} * 1.0 = 3.99\%$.This corrected$\phi_{E2}$ is the forward direction of the flow, it shall be negative.

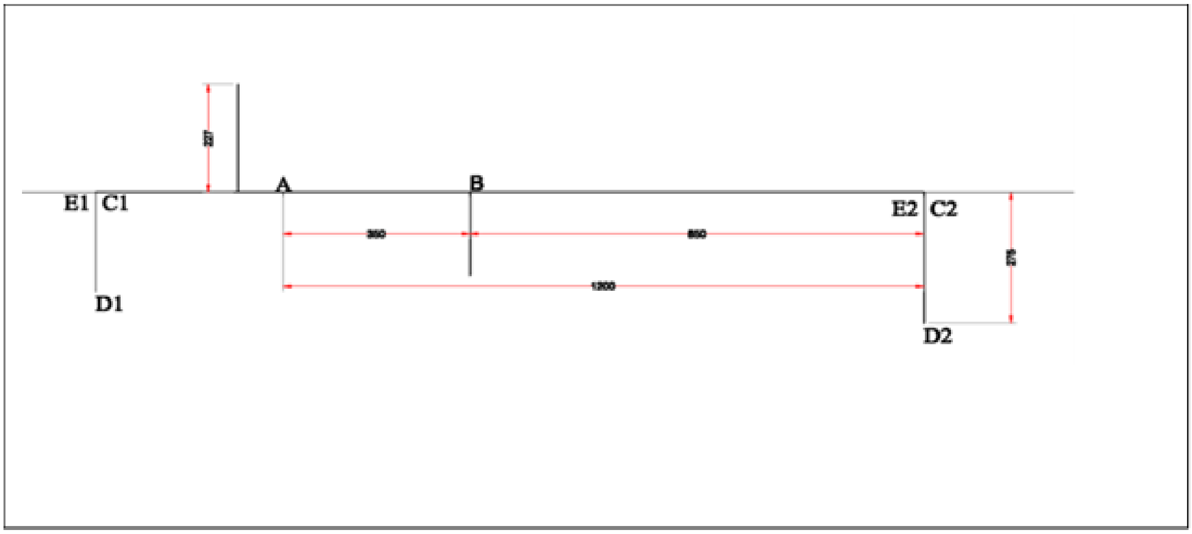
Correction interferences in d/s pile

- Effective length (b') =total length – sum of pile thickness=17-0.4*2=16.2m
- Total floor length = 17m
- Depth (D) of u/s pile =u/s cut off depth –t @u/s impervious apron = 2-0.50= 1.5m
  D (depth of the d/s depth) =d/s cut off depth –t @ 6m from toe =2.75-1.0=1.75m

Correction = (19*$\sqrt{\frac{D}{b'}}$)*$\frac{d+D}{b}$=19*$\sqrt{\frac{1.5}{16.2}}$ *($\frac{1.5+1.75}{17}$) =1.10% (-ve) because E_2 in the forward direction of flow, thus the correction shall be negative.

Corrected ϕ_{E2}=35.2-(3.99+1.10) =30.10%

∴The corrected pressures are ϕ_{c1}=73.26% and ϕ_{E2}=30.10%.



Finger 14: Pressure at key point

The unbalance pressure head

The length from point A to d/s is 12m

Pressure at point A =30.10+$\frac{73.26-30.10}{17}$*12=60.56%

Pressure head at point A (H) =$\frac{60.56*2.17}{100}$=1.31m

Thickness (t) =$\frac{H}{G-1}$ =$\frac{1.31}{2.3-1}$=1.01. < 1.5. It is safe.

Pressure at point B=30.10+$\frac{73.26-30.10}{17}$*8.5=54.22%

Pressure head at point B (H) =$\frac{54.22*2.17}{100}$=1.18m

Thickness (t) =$\frac{H}{G-1}$ =$\frac{1.18}{2.3-1}$=0.91. < 1. It is safe.

The impervious floor thickness is safe Khoslas theories

4.11. Stability Analysis of the Weir

a) Forces acting on the weir

The designed section has to be safe against sliding, overturning and tension crack. The followings are the major forces considered in the design of the weir overflow section by which the stability analysis was computed.

- Self-weight of the structure(W)
- External water pressure(Ph)
- Silt pressure(Ps)
- Up lift pressure

Structural damage due to seismicity is considered to be negligible. In the computation process of the stability analysis for the structure earth quake force is therefore assumed to be negligible

b) Self-weight of the structure

For the ease of calculating moment arm for each section of the curved profile of the ogee, the curved surface was assumed to be linear at proper intervals so that a trapezium section can be obtained. Now the total section of the weir was divided in to sub sections as shown in figure below.

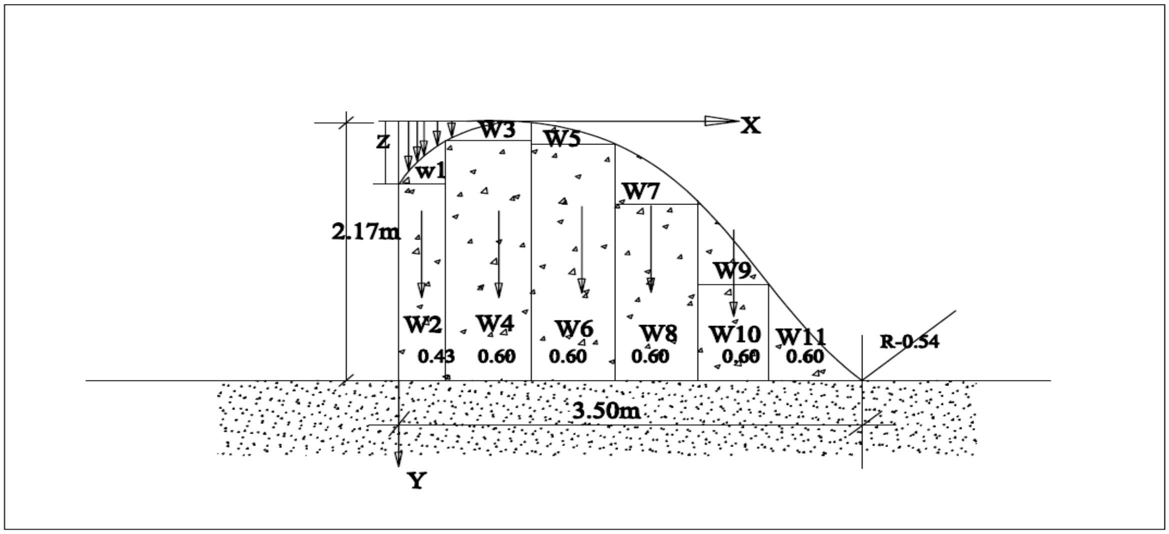


Figure 15: Self weight determination

c) Hydrostatic pressure

These are the forces acting on the weir due to the reservoir created upstream of the overflow section and the dynamic pressure created at the toe due to change in the momentum of the flow.

The external water pressure on the upstream face of the weir is calculated for sever case i.e for the design discharge level.

It has the four components P_w1, Pw2, Pw3 and P_w4 as shown in fig below. The water pressure that could be exerted on the weir body due to a change in momentum as the water flows over the curved toe surface was also calculated and incorporated in the analysis.

This is calculated based on the following formula

- $Pw1 = \gamma w \frac{h_1^2}{2}$, Acting at $h\left[\frac{h_1}{3}\right]$ in KN/m
- Where = h_1 is the weir height.

And for the pressure at u/s curved surface, $(Z), (Pw2) = \gamma W * Z * b \; in \; KN/m$

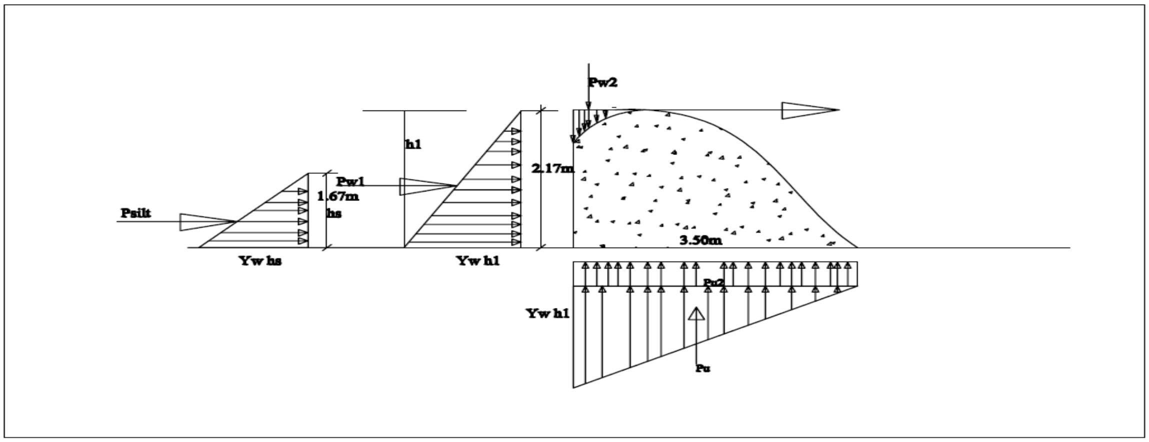


Figure 16: Static water pressure

From the above figure14 for dynamic case additional pressure at toe will be calculated by the following formula

Tail water pressure $(Pw3) = \gamma w \frac{h_2^2}{2}$

Water pressure at the d/s curved surface (P_w4) = 1.08*γw*h_2 where 1.08 is the width of pounded water

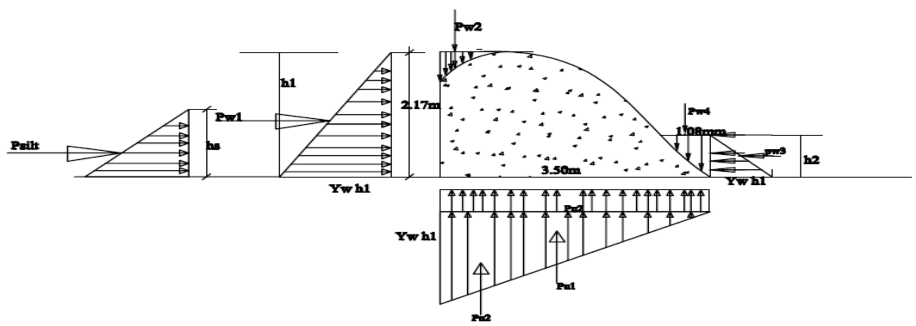


Figure 17: Dynamic case for hydrostatic, silt and uplift pressure

d) Silt pressure

The gradual accumulation of significant deposit of fine sediments especially silt, against the face of the weir generates a result of horizontal pressure Ps on the upstream section of the weir. Its magnitude is a function of the sediment depth at worst condition with a height equals to silt height (h_s).

The silt pressure is computed using the widely used Rankin's formula.

$$Psilt = gs * hs2 * (1 - sin\Phi)/(1 + sin\Phi))/2 \qquad (4.15)$$

- P_{silt}= Silt pressure
- gs= the unit weight of the silt
- h= the height of the silt to be deposited
- Φ=angle of internal friction=30°

e) Uplift pressure (Pu)

Equilibrium seepage patterns will develop under a weir section due to pores or discontinuous.

It is given by

- Pond level case
- Pu =γw$\frac{h_1*B}{2}$where B= bottom width of the weir, refer figure 13
- Dynamic case
  - $P_U1=\gamma_w*h_2*B$ acts at B/2 from the toe as shown figure 14
  - $P_U 2 =0.5*B*\gamma_w*(h_1-h_2)$

Where: - h1 water depth in u/s and

h_2 tail water depth.

The stability analysis is done for expected sever different load combinations. This is the condition when the weir body is subjected to design flood water and pond levels with all intakes and sluice gates are closed, tail water depth at the downstream level and silt pressure equivalent to the silt height is acting on the upstream face of the overflow section.The conventional assumptions for the stability analysis are as shown the table below

Table 15 Sign Convention

Vertical Force	**Horizontal Force**	**Moments @Point O**
Downward = (+ve)	Towards U/S = (+ve)	Anticlockwise = (+ve)
Upward = (-ve)	Towards D/S = (-ve)	Clockwise = (-ve)

Table 16 Unit weight of materials

Unit weight of materials		Values
Stone masonry		22KN/m^3
Mass concrete		23KN/m^3
Reinforced concrete		24KN/m^3
γ_{water}		10KN/m^3
$\gamma_{saturated}$		18KN/m^3
$\gamma_{sumerged}$		8KN/m^3

- **Overturning**
  - Fo = Sum of stabilizing moments / Sum of overturning moments
  - Fo >= 1.5
- **Sliding**
  - Fs=μ*(ΣV/ΣH)
  - μ=0.65-0.75,
  - Fs>=2

- **Shear friction Factor**
  - $SFF=((\mu*\Sigma V)+bq)/\Sigma H$
  - b= width of the wear (not crest length)
  - q=Shear strength between rock and weir=1400KN/m^2
  - $SFF >= 5$
  - if Fs < 2 and SFF >=5 safe against sliding
- **Tension**
  - if e<=B/6 no tension
- **Compression (crushing)**

1. Vertical stress when the reservoir is full

   (i) At heel $fyu=\Sigma V/B(1-(6e/B))$

   (ii) At toe $fyd=\Sigma V/B(1+(6e/B))$

   $e=M/\Sigma V$

2. Vertical stress when the reservoir is empty

   (i) At toe $fyd=\Sigma V/B(1-(6e/B))$

   (ii) At heel $fyu=\Sigma V/B (1+(6e/B))$

Generally if e<B/6 toe and heel in compression if e=B/6 fyd=0 at the heel

Principal stress

I. Principal stress when the reservoir is full
   A. Vertical stress $\sigma d=fyd \sec\varphi d^2-(p'-pe') \tan^2\varphi d$
      - p'= water pressure at the toe=H*W
      - pe'= earthquake pressure
   B. Shear stress $\tau d=(fyd- (p'-pe')) \tan\varphi d$ (the direction is towards upstream)

II. Principal stress when the reservoir is empty
   A. Vertical stress $\sigma u=fyu \sec\varphi u^2-(p'-pe') \tan^2\varphi u$
      - p'= 0=water pressure at the heel=H*W
      - Pe'= earthquake pressure
   B. Shear stress $\tau u=-(fyu- (p'-pe')) \tan\varphi u$ (the direction is towards downstream) Allowable stress should not above 7MPa

To find e: set ΣM about the toe

$X=\Sigma M/\Sigma V$ ………………………………………………….…. (4.16)

Where x is the moment arm of the resultant force from the center of the base

$$e = (B/2) \ldots \ldots \quad (4.17)$$

Table 17: Stability analysis of weir

Type Of Load (Pressure)		Vertical Load	Horizontal Load		Moment Arm	Moments Point O	
		+ve	+ve	-ve		+ve	-ve
	W1	2.13			3.21	6.83	
	W2	17.21			3.29	56.53	
	W3	0.97			2.87	2.77	
Self-weight	W4	28.01			2.77	77.6	
	W5	2.48			2.27	5.64	
	W6	23.05			2.17	50.01	
	W7	3.8			1.67	6.34	
	W8	15.46			1.57	24.27	
	W9	5.11			1.07	5.46	
	W10	5.24			0.97	5.09	
	W11	0.22			0.61	0.13	
	W12	1.06			0.59	0.62	
	W13	1.55			0.18	0.28	
Silt pressure			7.36		0.56		6.92
✓ Water pressure and Uplift pressure for two cases 1. Pond level case ❖ Weir height(h)=2.17, height of u/s curved surface(Z) =0.43							
Water pressure	P_w1			23.54	0.72		17.03
	P_w2	0.93			3.36	3.1	
Uplift pressure	U_p				2.33		88.61
Sum of forces		70.1	26			244.67	112.56
2. Dynamic (flood) case ❖ Tail water height(Y1)=h2 =0.91and Tail water width (a) =1.08							
Water pressure	P_w1			23.54	0.72		17.03
	P_w2	0.93			3.36	3.1	
	P_w3		4.05		0.3	1.22	
	P_w4	4.9			0.36	1.74	
Uplift pressure	U_P1				1.75	49.33	
	U_P2				2.33	47.48	
Sum		60.8	26			244.76	112.56

Summation of forces and moment for

1. Pond level case

- Sum of vertical force = 70.01KN/m
- Sum of horizontal force =26KN/m
- Sum of stabilizing moment ($M^{+)}$ =244.67KNm/m
- Sum of overturning moment ($M^{-)}$ =112.56KNm/m

Factor of Safety

- ❖ Factor of safety against overturning (F_O): the factor of safety against overturning should not be less than 1.5.

$$F_o = \frac{\sum M^+}{\sum M^-} = \frac{244.67}{112.56} = 2.2 > 1.5 \text{ it } \textbf{is safe.}$$

- ❖ Factor of safety against Sliding (F_s) = $\eta * \frac{\sum V_f}{\sum H_f}$ the valve of η range from 0.65- 0.75.for calculation we take 0.75

$$=0.75*\frac{70.01}{34} = 1.68 > 1.5 \textbf{safe!}$$

- ❖ Tension, by the Middle third rule if e<B/6 no tension

Location of the resultant force from the toe,

$$\bar{X} = \frac{\sum M_{net}}{\sum V_f} = \frac{244.67 - 112.56}{107.19 - 37.16} = 1.95$$

Eccentricity, e = 0.5*B - $\bar{X}$= 0.5*3.5-1.87 = -0.2m. Negative sign eccentricity is the resultant force acts near the heel.

- ❖ Eccentricity, e=-0.2m < $\frac{B}{6}$ = 3.5/6=0.583m (no tension)

Dynamic high flood case

- ❖ Sum of vertical force = 60.8KN/m

- Sum of horizontal force = 26KN/m
- Sum of stabilizing moment (M^{+}) =244.76KNm/m
- Sum of overturning moment (M^{-}) =122.69KNm/m

(i) Factor of safety against overturning(F_O):

$$F_o = \frac{\sum M^{+}}{\sum M^{-}} = \frac{247.62}{122.69} = 1.95 > 1.5 \text{ it } \textbf{is safe}$$

(ii) Factor of safety against Sliding

$$(F_s) = \eta * \frac{\sum V_f}{\sum H_f} = 0.75*\frac{60.8}{26} = 1.7 > 1.5 \textbf{ safe}$$

(iii)Tension

Location of the resultant force from the toe,

$$\bar{X} = \frac{\sum M_{net}}{\sum V_f} = 1.95$$

Eccentricity,

$$e = -0.2m < \frac{B}{6} = 3.5/6 = 0.583m \text{ (no tension) i.e. the resultant force acts near the heal}$$

The stability analysis shows that the proposed weir section is structurally stable. So, provide weir body of dimension. Bottom width = 3.50m; Height =2.17m

4.12. Weir component Design

The divide wall separates the under sluice from the main weir portion and allows a silt free water flow to the head regulator by depositing the silt in the under sluice pocket.

(i) Divide Wall

Design consideration

- Silt pressure occurring on the u/s divide wall when the river is at low flow.
- Differential head of water when the sluice gate is closed and flow over the weir.

- The critical case for the determination of wall height is when there is maximum flood

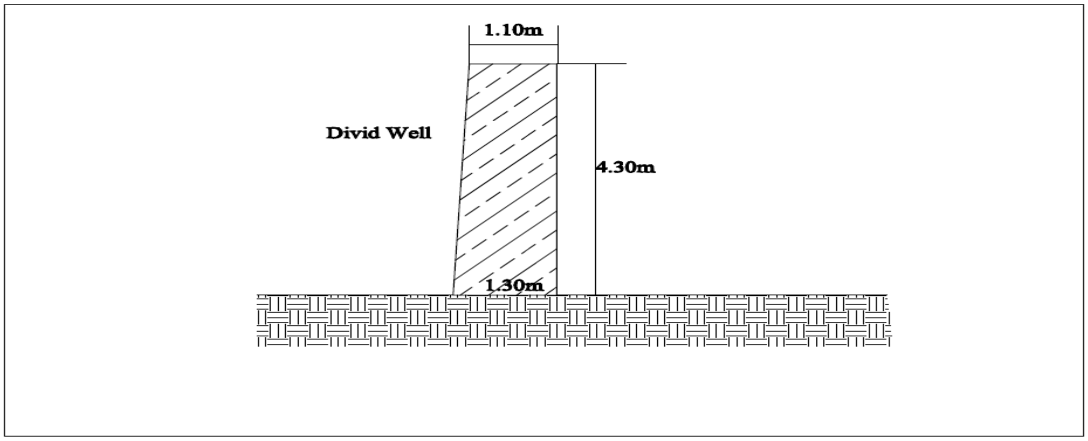


Figure 18: Simple sketch of divide wall

Assume divide wall thickness bottom 1.30m and slab top 1.10m & length 0.5m

A) u/s divide wall

U/s HFL = 1941.933

U/s bed level = 1938.25 and Adding 0.6m freeboard

Divide wall height= U/SHFL- river bed level + free board

= 1981.933-1938.25+0.6 =4.3

(ii) Stability Analysis of Divide Wall

The force components of this structure are its self-weight, silt pressure, Uplift pressure and water pressures

Available data:

- $\gamma_{concrete}$ = 24 KN/m^3,
- γ_{water} = 10 KN/m^3,
- $\gamma_{saturated}$ = 18KN/m^3
- $\gamma_{submerged}$= 8KN/m^{3}and
- $\gamma_{masonry}$= 23KN/m^3
    - Φ = 30^0

- Ka $=\frac{1-sin\phi}{1+sin\phi}=\frac{1-sin30}{1+sin30}$ =0.33

Table 18: Divide wall stability

Type of load	Vertical load KN/m		Horizontal load KN/m	moment KN_m/m	
	↓	↑	→	Stabilize	Over turning
Divide wall weight (w1)	134.16			87.204	
W2 of slab& breast wall	12.4				3.1
silt pressure			2.2		1.23
water pressure			23.54		17.03
uplift		14.105			3.6167
sum	146.56	14.105	25.75	87.204	26.06

I. Factor of safety against overturning

$$\text{Fo}=\frac{\text{resisting moment}}{\text{overturning moment}}>1.5$$

$=\frac{87.209}{26.059}=3.35>1.5$ it is Safe against overturning

II. Factor of safety against sliding

$$\text{Fo}=\frac{\mu*\text{summation ofstabilizing force}}{\text{summation of over turnig force}}=1.5$$

$=\frac{0.75*104.75}{25.75}=2.034$>1.5 ∴ Safe against sliding

III. Checking for tension $\bar{X}=\frac{summationofmoments}{summationofverticalforce}$

$=\frac{61.1447}{132.45}=$ 0.4616

Eccentricity (e) $=\frac{B}{2}-\bar{X}=\frac{1.3}{2}-0.4616=0.188$

Now compare (e) and $\frac{B}{6}=0.2167$∴o NO Tension (iii)Under sluice

Some of the important roles that the under sluice plays are;

- Enables the canal to flow silt free water from surface as much as possible
- Scour the silt deposited in front of the canal off take (regulator)
- Preserve a clear and defined river channel approaching the regulator.

The sill level of this under sluice is fixed to be 0.77 m higher than the minimum river bed level.

Hence the sill level of the under sluice=1938.25+0.77=1939.02maslwhere the canal sill level is 1940.02masl

Considering this, the opening size of the gate is 0.6 m*0.4 m. Considering Orifice (opening or vent) flow and pond level case, the discharge passing is computed using the following formula.

Q=Cd*L*H*(2*g*h) ^0.5

Q= 0.51m^3/sec

The under sluice can discharge 0.51m^3/sec which is more than two times of the head regulator. Hence, during non-rainy time, it is possible to flush the silt easily when required.

Hydrostatic force exerted on sheet opening.

- Water depth=0.77+0.6=1.37 m
- Width of the opening=0.4m

Hydro Static water Pressure for head of 0.6 m at the bottom of the gate = 6 KN/m^2

The critical case in the case of under sluice is during non-flow condition. The high flood condition is expected during summer. In this condition, water is not required for irrigation. If water is not required for irrigation, the under sluice should be fully opened

(iv)Gate for the under sluice

The gate for under sluice is to be vertical sheet metal of size 0.6m x 0.4m for the closure of the opening space providing some extra dimensions for the groove insertion. Gross area of sheet metal for the gate will be 0.65m x 0.45m (allowing 5cm insertion for Grooves).

The grooves are to be provided on the walls using angle iron frames at the two sides of the gate opening.

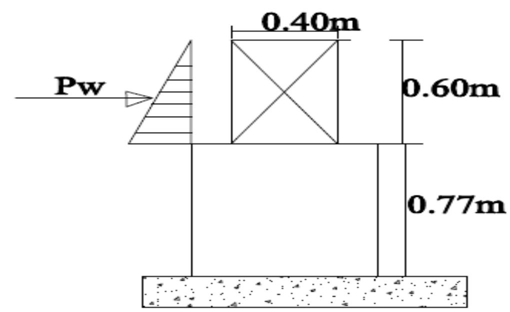


Figure 19: Gate for under sluice

Approximate weight of gate, G=0.706*(B^2*h*H) ^0.7

Where B=gate span (m)

h =gate height (m)

H=head of water over the sill (U/s HFL-Sill level =1941.933-1939.02=2.913m

Approximate weight of gate=0.26KN

Allowable tensile & bending stress of steel during wet condition =0.45*300=135 N/mm2 =13500N/cm2

Allowable tensile & bending stress of steel during wet condition
Hence bending stress in flat plate should be, $\delta=K*P*a^2/(100*S^2)$
Where S=thickness of the sheet metal (cm)
P=Hydrostatic pressure (N/Cm)=0.6N/cm^2, K=Non-dimensional facto a&b=gate width which related with K, a=0.4m
For plate aspect ratio b/a=0.6/0.4=1.5, K=52 from the table for different supporting condition.

S= (K*P*a^2/ (100*δ)) ^0.5= (52 *0.6*40^2/ (13500*100)) ^0.5=0.20cm

Considering incoming boulders and transported materials, take S=0.4 cm

(v) Head Regulator /Canal out Let

The head regulator is a structure at the head of an off taking canal from a reservoir behind a weir. It is provided to regulate supply in the canal control the entry of silt in the canal Shut out the river floods Based on the recorded base flow of 0.24m3/sec the size of fixed using;

Q=cd*(2*9.81) ^0.5*L*H

Where H= water depth, Cd=0.62

The capacity of the regulator has at least the lean flow and the gate dimensions are determined as water depth H=0.45m and width 0.35m

The canal sill level is weir crest level-water depth, 1940.2-0.35=1939.85masl

Water pressure on the gate is $P = {}^{\gamma}w \times waterheight = 10 * 1.37 = 13.7KN/m^2$

S= (K*P*L^2/ (100*δ)) ^0.5

For H/L 0.45/0.35=1.3 K value will be 44.3

$$S = \frac{K \times P \times L^2}{100 \times \delta} = \left(\frac{44.3 \times 1.37 \times 35^2}{100 \times 13500}\right)^2 = 0.132cm$$

Considering incoming boulders and transported materials, take S=0.2 cm

(vi) Stilling Basin

The transition from super critical to subcritical flow takes place in the form of hydraulic jump. The stilling basin is designed to insure that the jump occurs always at such location that the flow velocity entering the erodible downstream channels are incapable causing harmful scour. The design of a particular stilling basin is depend on the magnitude of Froude number and other characteristics of flow to be handled. For the design discharge a Froude number and sequent depths are as follow. Fr = 2.188, Y1=0.912m& Y2=2.578m

For Froude number between 2.5 & 4.5, type (I) stilling basin is selected. Taken from hydraulic structure Chute block height = 2*Y1=1.824m,

Length of chute = 2*Y1=1.824m,

End sill height = 1.25*Y1=1.14m,

Space between blocks = 2.5*Y1=2.28m,

Length of basin=11.57m

Table 19: stilling basin length and Froude number relation

Fr	2	2.188	3
L/Y2	4.3	4.488	5.3
L	11.0854	11.57	13.6634

4.13. Bill of Quantity

Table 20: BOQ analysis

ITEM No.	ACTIVITY DISCRIPTION A HEAD WORK STRACTURE	UNIT	QUANTITY	UNIT COST	TOTAL COST
1	weir body				
1.1	Earth Excavation	m^3	169.2	35	5922
1.2	hard rock excavation	m^3	148.05	200	29610
1.3	Backfill	m^3	50.5	25	1682.155
1.4	Lean Concrete (C10)	m^2	211.5	950	200925
1.5	Masonry	m^3	272.052	120	32646.24
1.6	Reinforced Concrete				
1.6.1	Concrete (C20)	m^3	181.368	1560	282934.08
1.6.2	cyclopean concrete	m^3	152.2	2000	304400
1.6.3	Reinforcing bars ф 12	kg	1644.3	35	57550.5
	Sub Total				**915,670.00**
2.1	Earth Excavation				
2.2	Hard rock excavation	m^3	18.8	200	3760
2.3	lean concrete	m^2	69	950	65550
2.4	Concrete (C20)	m^3	34.5	1560	53820
2.5	Backfill	m^3	11.75	25	293.75
	Sub Total				**124080.528**
2.1	Earth Excavation	m^3	28.2	23.29	656.778
3	Downstream apron				
3.1	Earth Excavation	m^3	282	23.29	6567.78
3.2	Hard rock excavation	m^3	282	200	56400
3.3	lean concrete	m^2	552	950	524400
3.4	Concrete (C20)	m^3	564	1598.82	901734.5
3.5	Backfill	m^3	47	33.31	1565.57
	Sub Total				**148,9102.00**
4	upstream pile				
4.1	hard rock excavation	m^3	98.7	200	19,740
4.2	lean concrete	m^2	18.8	950	17,860
4.3	concrete (C20)	m^3	28.2	1560	43,992.00

ITEM No.	ACTIVITY DISCRIPTION A HEAD WORK STRACTURE	UNIT	QUANTITY	UNIT COST	TOTAL COST
4.4	Reinforcing bars ф 12	Kg	26.25	35	918.75
4.5	backfill	m^3	70.5	25	1,762.50
	Sub Total				**84,273.25**
5	downstream pile				
5.1	Hard rock excavation	m^3	250.60	200	50,120.00
5.2	lean concrete	m^2	18.8	950	17,860.00
5.3	Concrete (C20)	m^3	96.30	1560	150,228.00
5.4	Reinforcing bar	kg	21.30	36	766.80
5.5	Backfill	m^3	82.25	25	2,056.25
	Sub Total				**221,031.05**
6	Dived wall				
6.1	Masonry	m^3	8.385	80	670.80
6.2	Concrete (C20)	m^3	5.59	1560	8,720.40
6.3	lean concrete	m^2	3.25	950	3,087.50
6.4	Reinforcing bars ф 12	kg	44.36	35	1,552.60
6.5	Plastering	m^2	3.25	150	487.50
	Sub Total				**14,518.80**
7	Under sluice left side				
7.1	Masonry	m^3	0.20	80.00	16.00
7.2	Lean Concrete (C10)	m^2	0.40	950.00	380.00
7.3	Reinforced Concrete	kg	0.00	0.00	0.00
7.4	Concrete (C20)	m^3	0.10	1560	156.00
7.5	Reinforcing bars ф 10	Kg	574.90	22.00	12,647.80
	Sub Total				**13,199.80**
8	Upstream retaining wall for both left and right side				
8.1	earth Excavation	m^3	45.6	25	1140
8.2	Hard rock excavation	m^3	19.25	200.00	3,850.00
8.3	lean concrete	m^2	30.00	950.00	28,500.00
8.4	Masonry	m^3	77.40	80.00	6,192.00
8.5	Concrete (C20)	m^3	51.60	1560	80,496
8.6	Backfill		23.65	25	591.25
	Sub Total				**120,769.25**
	Total Cost				**915,669.98**

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The Asher Diversion weir design Project has about140ha irrigable. From this command area springs, school and other unfavorable conditions for irrigation found, thus around 130ha is irrigated. Even though, the above factors decrease the irrigable area, the project increases traditional irrigable area of 15ha around nine times.

The ogee type weir is selected in order to dissipate the higher energy due to higher discharge and boulders that comes from river flow. It is structurally safe but the design analysis and construction of ogee type is difficult as compared to broad crested weir type. In addition to this, divide wall thickness is 1.3m to be safe structurally.

The upstream divide wall is provided 0.50m top width with 3m bottom width to be structurally safe. Downstream divide wall dimension is 0.40m top width and 2.50m bottom width. The bank of the river is not much safe. Hence, design10m for upstream and 16m for downstream.

The stilling basin is designed to insure that the jump occurs always at such location that the flow velocity entering the erodible downstream channels are incapable causing harmful in this case the Froude number between 2.5 and 4.5, Type I stilling basin is selected.

5.2. Recommendation

During implementation period, water diversion system has to be done in a very professional manner as the recommendations and design.

- Construct the canal and under sluice outlet up to the head regulator with surrounding retaining walls.
- Divert the base flow to the under sluice channel
- Construct the weir body.
- Don’t try to work in rainy seasons as there is excess run off to the river.

In the Recommendations that the following activities will be carried out by the farmers regularly to ensure that the scheme is in proper condition to serve the intended purpose

- Flush out the accumulated silt
- Stir up the accumulated silt
- Clean the trash screens
- Grease the movable metal parts
- All the concrete and steel parts with defects should be repaired

6. REFERENCE

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7. Appendices

Table 1.6 Antecedent Rainfall Conditions and Curve Numbers (for I_a=0.2S)

Curve Number for Condition II	**Factor to Convert Curve number for Condition II to**	
	Condition I	**Condition III**
10	0.40	2.22
20	0.45	1.85
30	0.50	1.67
40	0.55	1.50
50	0.62	1.40
60	0.67	1.30
70	0.73	1.21
80	0.79	1.14
90	0.87	1.07
100	1.00	1.00

Appendices. 2

TABLE 12.3.1
K_T values for Pearson Type III distribution (positive skew)

Skew coefficient C_s or C_w	Return period in years						
	2	5	10	25	50	100	200
	Exceedence probability						
	0.50	0.20	0.10	0.04	0.02	0.01	0.005
3.0	–0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	–0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	–0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	–0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	–0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	–0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	–0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	–0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	–0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	–0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	–0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	–0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	–0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	–0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	–0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	–0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	–0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	–0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	–0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	–0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	–0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	–0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	–0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	–0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	–0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	–0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	–0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	–0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.2	–0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	–0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0	0.842	1.282	1.751	2.054	2.326	2.576

TABLE 12.3.1 (*cont.*)
K_T values for Pearson Type III distribution (negative skew)

Skew coefficient C_s or C_w	Return period in years: 2	5	10	25	50	100	200
	Exceedence probability: 0.50	0.20	0.10	0.04	0.02	0.01	0.005
−0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
−0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388
−0.3	0.050	0.853	1.245	1.643	1.890	2.104	2.294
−0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
−0.5	0.083	0.856	1.216	1.567	1.777	1.955	2.108
−0.6	0.099	0.857	1.200	1.528	1.720	1.880	2.016
−0.7	0.116	0.857	1.183	1.488	1.663	1.806	1.926
−0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
−0.9	0.148	0.854	1.147	1.407	1.549	1.660	1.749
−1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
−1.1	0.180	0.848	1.107	1.324	1.435	1.518	1.581
−1.2	0.195	0.844	1.086	1.282	1.379	1.449	1.501
−1.3	0.210	0.838	1.064	1.240	1.324	1.383	1.424
−1.4	0.225	0.832	1.041	1.198	1.270	1.318	1.351
−1.5	0.240	0.825	1.018	1.157	1.217	1.256	1.282
−1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
−1.7	0.268	0.808	0.970	1.075	1.116	1.140	1.155
−1.8	0.282	0.799	0.945	1.035	1.069	1.087	1.097
−1.9	0.294	0.788	0.920	0.996	1.023	1.037	1.044
−2.0	0.307	0.777	0.895	0.959	0.980	0.990	0.995
−2.1	0.319	0.765	0.869	0.923	0.939	0.946	0.949
−2.2	0.330	0.752	0.844	0.888	0.900	0.905	0.907
−2.3	0.341	0.739	0.819	0.855	0.864	0.867	0.869
−2.4	0.351	0.725	0.795	0.823	0.830	0.832	0.833
−2.5	0.360	0.711	0.771	0.793	0.798	0.799	0.800
−2.6	0.368	0.696	0.747	0.764	0.768	0.769	0.769
−2.7	0.376	0.681	0.724	0.738	0.740	0.740	0.741
−2.8	0.384	0.666	0.702	0.712	0.714	0.714	0.714
−2.9	0.390	0.651	0.681	0.683	0.689	0.690	0.690
−3.0	0.396	0.636	0.666	0.666	0.666	0.667	0.667

Source: U. S. Water Resources Council (1981).

Appendix. 3

OGL	change	CBL	CFSL	HFL	Conc.CBL	Conc. CFSL	Conc. HFL	Conc .OGL	Elev. Diff
1940.6	0	1940.2	1940.6	1940.8	0,1940.2	0,1940.6	0,1940.8	0,1940.6	0.4
1939.4	50	1940.1	1940.5	1940.7	50,1940.1	50,1940.5	50,1940.7	50,1939.4	-0.7
1938.8	100	1940	1940.4	1940.6	100,1940	100,1940.4	100,1940.6	100,1938.8	-1.2
1939.25	150	1939.9	1940.3	1940.5	150,1939.9	150,1940.3	150,1940.5	150,1939.25	-0.65
1939.87	200	1939.8	1940.2	1940.4	200,1939.8	200,1940.2	200,1940.4	200,1939.87	0.07
1940.79	250	1939.7	1940.1	1940.3	250,1939.7	250,1940.1	250,1940.3	250,1940.79	1.09
1940.85	300	1939.6	1940	1940.2	300,1939.6	300,1940	300,1940.2	300,1940.85	1.25
1940.65	350	1939.5	1939.9	1940.1	350,1939.5	350,1939.9	350,1940.1	350,1940.65	1.15
1938.26	400	1939.4	1939.8	1940	400,1939.4	400,1939.8	400,1940	400,1938.26	-1.14
1938.68	450	1939.3	1939.7	1939.9	450,1939.3	450,1939.7	450,1939.9	450,1938.68	-0.62
1937.62	500	1939.2	1939.6	1939.8	500,1939.2	500,1939.6	500,1939.8	500,1937.62	-1.58
1937.21	550	1939.1	1939.5	1939.7	550,1939.1	550,1939.5	550,1939.7	550,1937.21	-1.89
1937.87	600	1939	1939.4	1939.6	600,1939	600,1939.4	600,1939.6	600,1937.87	-1.13
1939.52	650	1938.9	1939.3	1939.5	650,1938.9	650,1939.3	650,1939.5	650,1939.52	0.62
1938.85	700	1938.8	1939.2	1939.4	700,1938.8	700,1939.2	700,1939.4	700,1938.85	0.05
1938.9	750	1938.7	1939.1	1939.3	750,1938.7	750,1939.1	750,1939.3	750,1938.9	0.2
1937.95	800	1938.6	1939	1939.2	800,1938.6	800,1939	800,1939.2	800,1937.95	-0.65
1939.95	850	1938.5	1938.9	1939.1	850,1938.5	850,1938.9	850,1939.1	850,1939.95	1.45
1936.83	900	1938.4	1938.8	1939	900,1938.4	900,1938.8	900,1939	900,1936.83	-1.57
1936.85	950	1938.3	1938.7	1938.9	950,1938.3	950,1938.7	950,1938.9	950,1936.85	-1.45
1937.61	1000	1938.2	1938.6	1938.8	1000,1938.2	1000,1938.6	1000,1938.8	1000,1937.61	-0.59
1937.57	1050	1938.1	1938.5	1938.7	1050,1938.1	1050,1938.5	1050,1938.7	1050,1937.57	-0.53
1939.5	1100	1938	1938.4	1938.6	1100,1938	1100,1938.4	1100,1938.6	1100,1939.5	1.5
1938.45	1150	1937.9	1938.3	1938.5	1150,1937.9	1150,1938.3	1150,1938.5	1150,1938.45	0.55
1939.6	1200	1937.8	1938.2	1938.4	1200,1937.8	1200,1938.2	1200,1938.4	1200,1939.6	1.8
1938.28	1250	1937.7	1938.1	1938.3	1250,1937.7	1250,1938.1	1250,1938.3	1250,1938.28	0.58
1934.65	1300	1937.6	1938	1938.2	1300,1937.6	1300,1938	1300,1938.2	1300,1934.65	-2.95
1939.65	1350	1937.5	1937.9	1938.1	1350,1937.5	1350,1937.9	1350,1938.1	1350,1939.65	2.15
1938.5	1400	1937.4	1937.8	1938	1400,1937.4	1400,1937.8	1400,1938	1400,1938.5	1.1
1935.85	1450	1937.3	1937.7	1937.9	1450,1937.3	1450,1937.7	1450,1937.9	1450,1935.85	-1.45
1935.75	1500	1937.2	1937.6	1937.8	1500,1937.2	1500,1937.6	1500,1937.8	1500,1935.75	-1.45
1935.65	1550	1937.1	1937.5	1937.7	1550,1937.1	1550,1937.5	1550,1937.7	1550,1935.65	-1.45
1935.5	1600	1937	1937.4	1937.6	1600,1937	1600,1937.4	1600,1937.6	1600,1935.5	-1.5
1935.75	1650	1936.9	1937.3	1937.5	1650,1936.9	1650,1937.3	1650,1937.5	1650,1935.75	-1.15
1935.5	1700	1936.8	1937.2	1937.4	1700,1936.8	1700,1937.2	1700,1937.4	1700,1935.5	-1.3
1935.5	1750	1936.7	1937.1	1937.3	1750,1936.7	1750,1937.1	1750,1937.3	1750,1935.5	-1.2
1935.5	1800	1936.6	1937	1937.2	1800,1936.6	1800,1937	1800,1937.2	1800,1935.5	-1.1

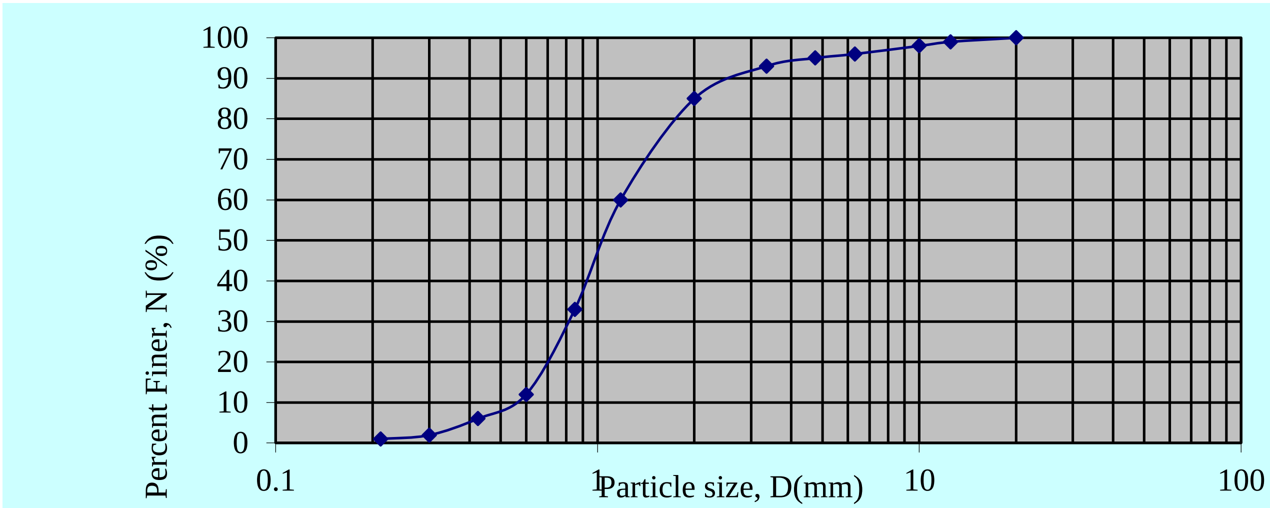
1938.97	1850	1936.5	1936.9	1937.1	1850,1936.5	1850,1936.9	1850,1937.1	1850,1938.97	2.47
1938.25	1900	1936.4	1936.8	1937	1900,1936.4	1900,1936.8	1900,1937	1900,1938.25	1.85
1935.6	1950	1936.3	1936.7	1936.9	1950,1936.3	1950,1936.7	1950,1936.9	1950,1935.6	-0.7
1934.5	2000	1936.2	1936.6	1936.8	2000,1936.2	2000,1936.6	2000,1936.8	2000,1934.5	-1.7
1935.83	2050	1936.1	1936.5	1936.7	2050,1936.1	2050,1936.5	2050,1936.7	2050,1935.83	-0.27
1934.9	2100	1936	1936.4	1936.6	2100,1936	2100,1936.4	2100,1936.6	2100,1934.9	-1.1
1937.8	2150	1935.9	1936.3	1936.5	2150,1935.9	2150,1936.3	2150,1936.5	2150,1937.8	1.9
1936.95	2200	1935.8	1936.2	1936.4	2200,1935.8	2200,1936.2	2200,1936.4	2200,1936.95	1.15
1936.53	2250	1935.7	1936.1	1936.3	2250,1935.7	2250,1936.1	2250,1936.3	2250,1936.53	0.83
1933.74	2300	1935.6	1936	1936.2	2300,1935.6	2300,1936	2300,1936.2	2300,1933.74	-1.86
1936.85	2350	1935.5	1935.9	1936.1	2350,1935.5	2350,1935.9	2350,1936.1	2350,1936.85	1.35
1936.23	2400	1935.4	1935.8	1936	2400,1935.4	2400,1935.8	2400,1936	2400,1936.23	0.83
1933.74	2450	1935.3	1935.7	1935.9	2450,1935.3	2450,1935.7	2450,1935.9	2450,1933.74	-1.56
1933.98	2500	1935.2	1935.6	1935.8	2500,1935.2	2500,1935.6	2500,1935.8	2500,1933.98	-1.22
1935.45	2550	1935.1	1935.5	1935.7	2550,1935.1	2550,1935.5	2550,1935.7	2550,1935.45	0.35
1934.9	2600	1935	1935.4	1935.6	2600,1935	2600,1935.4	2600,1935.6	2600,1934.9	-0.1
1934.24	2650	1934.9	1935.3	1935.5	2650,1934.9	2650,1935.3	2650,1935.50	2650,1934.24	-0.66

Appendix: 4

CUTOFF	FILL	partial distance	Area of cut	Volume of cut of cut	Area of fill	Volume of fill

0.4	-0.7	50	1.28	64	-2.2	-112
0.07	-1.2	50	0.224	11.2	-3.8	-192
1.09	-0.65	50	3.488	174.4	-2.1	-104
1.25	-1.14	50	4	200	-3.6	-182.4
1.15	-0.62	50	3.68	184	-2.0	-99.2
0.62	-1.58	50	1.984	99.2	-5.1	-252.8
0.05	-1.89	50	0.16	8	-6.0	-302.4
0.2	-1.13	50	0.64	32	-3.6	-180.8
1.45	-0.65	50	4.64	232	-2.1	-104
1.5	-1.57	50	4.8	240	-5.0	-251.2
0.55	-1.45	50	1.76	88	-4.6	-232
1.8	-0.59	50	5.76	288	-1.9	-94.4
0.58	-0.53	50	1.856	92.8	-1.7	-84.8
2.15	-2.95	50	6.88	344	-9.4	-472
1.1	-1.45	50	3.52	176	-4.6	-232
2.47	-1.45	50	7.904	395.2	-4.6	-232
1.85	-1.45	50	5.92	296	-4.6	-232
1.9	-1.5	50	6.08	304	-4.8	-240
1.15	-1.15	50	3.68	184	-3.7	-184
0.83	-1.3	50	2.656	132.8	-4.2	-208
1.35	-1.2	50	4.32	216	-3.8	-192
0.83	-1.1	50	2.656	132.8	-3.5	-176
0.35	-0.7	50	1.12	56	-2.2	-112
	-1.7	50			-5.4	-272
	-0.27	50			-0.9	-43.2
	-1.1	50			-3.5	-176
	-1.86	50			-6.0	-297.6
	-1.56	50			-5.0	-249.6
	-1.22	50			-3.9	-195.2
	-0.1	50			-0.3	-16
	-0.66	50			-2.1	-105.6

Appendix 5



100
90
80
70
60
50
40
30
20
10
0
0.1
1
10
100
Percent Finer, N (%)
Particle size, D(mm)